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## 17.1 SELECTION OF STRUCTURE TYPE

The selection of the proposed structure type is determined from evaluation of the Structure Survey Report with accompanying supplemental data, current construction costs, and preference based on past experience. Also in selecting the most economical structure ease of fabrication and erection, general features of terrain, roadway geometrics, subsurface exploration, and geographic location in the State of Wisconsin are considered. The proposed structure must blend into existing site conditions in a manner not detracting to its surrounding environment. Every attempt is made to select an aesthetically attractive structure consistent with structural requirements, economy, and geographic surroundings. See Chapter 4.

The economical span ranges of various types of structures are given in Figure 5.1 of this Manual. Superstructure span lengths are related to the cost of the substructure units. If the substructure units are relatively expensive, it is generally more economical to use the longer span lengths available for a given type structure. Practicality dictates using the average structure length for twin structures if the preliminary structure lengths are within 3 feet. A multiple span structure is made symmetrical if its end spans are within 3 feet in length of each other.

All structure span lengths are rounded-off to the nearest 6 inches except for stream crossings and multi span prestressed girder structures where the span lengths are adjusted to maintain equal girder lengths. For example, a typical multiple span prestressed girder structure has interior spans longer than its exterior spans. Refer to Standard 19.8 for details of girder lengths at abutment and piers.

On stream crossings, design structure span lengths to the nearest foot (center to center bearing) and skew angles in multiples of 5°. This is to standardize span lengths and skew angles.

For geometric considerations in structure selection reference is made to Chapter 3 of the Manual. The requirements for structure expansion and fixed pier locations are in Chapter 12 and bearing types are in Chapter 27 of this Manual. Also, expansion joint requirements and types are specified in Chapter 28 of this Manual. Since the skew angle for most snow plow blades is 35°, it is desirable to avoid this skew angle for bridge joints. This reduces the chances of joint damage resulting from the plow blades dropping into the expansion joints.

Standard Office details allow for construction tolerances during construction of the structure. It is the responsibility of the designer to allow for adjustments in connections during construction on large or complex structures where standard details are not adequate. Small fabrication errors, thermal movements, deflection movements, etc. can cause final field erection problems if there are no provisions for adjustment.

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**17.1A ALTERNATE STRUCTURE TYPES**

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Consider the following procedures when developing bridge plans:

1. Base preliminary plan development on an engineering and economic evaluation of alternate designs.
2. Evaluate alternative designs on the basis of competitive materials appropriate to a specific structure type.
3. Do not propose specific construction methods or erection procedures in the plans unless constraints are necessary to meet specific project requirements.
4. Make an economic evaluation of preliminary estimates based on state-of-the-art methods of construction for structure types.
5. Consider future structure maintenance needs in the structure's design in order to provide life cycle costing data.
6. Consider alternate plans where experience, expertise and knowledge of conditions clearly indicates that they are justified. Alternate plans are not compatible with stage construction and in these situations will not be used.
7. Value engineering concepts are recognized as being cost effective. Apply these concepts to the selection of structure type, size and location throughout the plan development process.

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## 17.2 SUPERSTRUCTURES

Superstructures are classified as Deck or Through types. In the deck type of structure, the roadway is above or on top of the supporting structures. In the through type of structure, the roadway passes between two elements of the superstructure, as a through type steel-truss or a tied-arch span. Through type superstructures are generally used where long span lengths are required. Deck type structures are predominant because they lend themselves to future widening if increased traffic requires it.

Specifications allow the distribution of parapets and sidewalk loads equally to all girders. However, large sidewalk loads should be placed and analyzed only on the exterior girder, as this more closely resembles the structure response. Some of the various types of superstructures used in Wisconsin are as follows:

(1) Concrete Slab (Flat, Haunched and Voids)

Concrete slab structures are adaptable to roadways with a high degree of horizontal curvature. This type of superstructure is functional for short to medium span lengths and is relatively economical to construct and maintain. The practical range of span lengths for concrete slab structures can be increased by using haunched slab structures. Concrete slab structures are limited to sites requiring a skew angle of 30 degrees or less. The voided slab structure is not being used due to excessive longitudinal cracking over the voids in the negative zone. Refer to Chapter 18.

(2) Prestressed Concrete Girder

Prestressed concrete girder structures are very competitive from a first cost standpoint and require very little maintenance. Manufacturing of prestressed girders can be conducted at the structure site or at a permanent location of the manufacturer. Future widening can be accomplished with comparative ease. Refer to Chapter 19.

(3) Concrete T-Beam

The concrete T-beam has had limited use in Wisconsin during recent years and is no longer used.

(4) Prestressed Concrete Slab or Box

Precast prestressed concrete slab or box structures have the advantage of rapid construction where traffic must be diverted. Elimination of the need for falsework is of particular advantage when vertical clearances are critical during the construction phase. Experience indicates that from a first cost standpoint, these structures are more expensive to construct than concrete slab structures. Refer to Chapter 19.

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(5) Concrete Box Girder

The concrete box girder structure is aesthetically adaptable for urban sites having roadways with a high degree of horizontal curvature as well as large skew angles. This structure is frequently employed in multi-level interchanges where horizontal clearances are limited as the pier cap is an integral part of the deck. Problems are encountered in maintenance with deck replacements requiring shoring.

(6) Concrete Rigid Frame

The concrete rigid frame is less economical than other superstructure types. However, the concrete rigid frame is known for its aesthetic value and is used most in public parks and urban areas where the span lengths are similar to concrete slab structures and approach embankments are relatively high.

(7) Steel Rolled Section and Welded Plate Girder

Welded plate girders are less expensive than rolled sections that require cover plates because of their reduced allowable design stress resulting from the fatigue criteria. Welded plate girders have greater versatility in allowing variable web thickness and depth as well as variable flange thicknesses. Future widening can be accomplished with comparative ease. Refer to Chapter 24 and Chapter 38 for Railroad Structures.

(8) Steel Box Girder

Steel box girder structures have span length capabilities similar to plate girders. Aesthetically they present a smooth, uncluttered appearance due to their closed box sections. Current experience shows that steel box girders require more material than conventional steel plate girders and need additional lateral bracing due to wind loads prior to deck placement.

(9) Steel Tied Arch and Steel Truss

Unusual bridge sites such as major river and harbor crossings may require the use of longer span lengths than conventional deck type superstructures can accommodate. This is where a steel arch or truss can be used effectively. Refer to Chapter 46.

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**(10) Timber Longitudinally Laminated Decks**

Timber structures blend well in natural settings and are relatively easy to construct with light construction equipment. Timber longitudinally laminated deck structures have low profiles that generally provide good clearances for high water. Their application is limited by the range of span lengths and economics in comparison to concrete slabs. Refer to Chapter 23.

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**17.3 DESIGN OF SLAB ON GIRDERS**

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**(1) General**

The design of concrete decks on prestressed or steel girders is based on AASHTO LRFD 4.6.2.1. Moments from truck wheel loads (one or two trucks side by side) are distributed over a width of deck which spans perpendicular to the girder. The width of deck or width of equivalent strip is given in Table 4.6.2.1.3-1 of AASHTO. Positive moments are distributed over a different deck width than negative moments.

The distribution width in inches for positive moments is equal to  $26 + 6.6 S$  and for negative moments is equal to  $48 + 3.0 S$  where  $S$  equals girder spacing in feet.

To minimize transverse deck cracking a minimum slab thickness of 8 inches is used for all decks on new bridges. For deck replacements a thinner deck may be used if a reduced dead load is required to increase live load capacity. Research on transverse deck cracking (NCHRP Report 297) recommends smaller diameter reinforcement to reduce transverse deck cracking. The maximum size of transverse bars used is #5 with a minimum spacing of 6.5 inches for decks of eight inches or more in thickness and 6 inches for decks less than 8 inches in thickness. Identical bar size and spacing is used for the top and bottom transverse bars with each layer offset  $\frac{1}{2}$  the bar spacing from the other. If top and bottom transverse bars align they form a weakened section within the concrete that is more susceptible to cracking.

For bridges with deck slabs on girders the most economical structure can be achieved by using as few lines of girders as possible. After the number of girders has been determined adjustments in girder spacing should be investigated to see if slab thickness can be minimized.

**(2) Two Course Deck Construction**

For skews of 20 degrees or greater the machine used to strike off and finish the concrete must have its longitudinal axis within 20 degrees of the center line of bearing of the substructure units. This produces more equal girder loads in a span during the concrete pour which results in dead load deflections being closer to the theoretical computed deflections. However, for steel girders with wide decks and large skews or continuous long span steel girders final dead load deflections may not be within a reasonable allowable variance from the theoretical. By using two course construction any discrepancies in deflections in the first pour can be corrected by varying the thickness of the second pour since most of the deflection will occur during the first pour.

When using two course construction the first pour is 1 inch less in thickness (1.5" bar cover) than the standard deck thickness and the second pour is a 2" minimum thickness Class E concrete overlay. For two course deck construction also add an

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additional 20 pounds for square foot for a future wearing surface. Also the top surface of the first pour is given a dragged or broom finish to obtain a roughened surface.

A report by the Kansas DOT entitled "Cracking and Chloride Content in Reinforced Concrete Bridge Decks" (Report No. K-Tran: KU-01-9) has determined that two course deck construction results in decks that have more severe cracking than monolithic decks. The report also states that the average chloride concentration at crack location exceeds the corrosion threshold by the end of the first winter season after construction. Some agencies specify a high density second course concrete overlay to provide a more durable riding surface but based on the findings of the Kansas report this practice should be avoided.

(3) Reinforcing Steel for Deck Slabs on Girders

A. Transverse Reinforcement

The live load moments used to determine the size and spacing of the transverse bars are shown in Table 17. These moments are from AASHTO LRFD Table A4-1 or from a continuous beam analysis based on three supports and no overhang. If the continuous beam analysis resulted in a moment which was greater than the moment in AASHTO Table A4-1, then the moment from Table A4-1 was used for design.

The continuous beam analysis is based on either one or two trucks, whichever gives the larger moment. Two trucks were placed side by side, four feet apart. Wheel loads were distributed over the tire width. The load factor used for two truck loading is 1.75. A multiple presence factor of 1.2 is used for one truck loading which results in a load factor of 2.1 ( $1.2 \times 1.75$ ).

Table 17 includes the multiple presence factor and impact but does not include the load factor of 1.75 for live load. The negative dead load moment over the support is determined from the formula:  $DLM = W * S * S/10$  where W is the uniform dead load of slab and wearing surface and S is the girder spacing. The positive dead load moment is determined from the formula:  $DLM = W * S * S/12.5$ . Negative moments at supports are adjusted to the moment at the location of the design section being considered.

Transverse bar steel requirements (bar size and spacing) are determined for both positive moment requirements and negative moment requirements and the same steel is used in both the top and bottom of slab. Figure 17.1 shows bar locations and clearances.

The "Distance from C/L of Girder to Design Section" is from AASHTO LRFD 4.6.2.1.6. For steel beams the distance is equal to one-quarter of the flange

width from the centerline of support. For prestress girders use the values in the following table.

Girder Depth (in.)	Distance (in.)
28	6
36	4
45	5
54W	15
70	10
72W	15
82W	15

The reinforcing steel in Table 17.1A and 17.2A does not account for deck overhangs. Check Table 17.3 or 17.4 to see the minimum amount of steel required in the overhangs. Also any concrete haunch over the girder flange is not considered in Table 17.1A or 17.2A.

The reinforcement shown in Tables 17.1A and 17.2A is based on both the strength requirement and crack control requirement. The tables are based on a concrete strength of 4 ksi and a bar steel strength of 60 ksi but the same tables should be used for concrete strength of 5 ksi. Two criteria were used to check crack control, AASHTO LRFD 5.7.3.4 and a proposed revision to 5.7.3.4.

- a) For crack control the steel stress from service moments cannot exceed the value from the following formula:

$$f_s = 130 / (d_c A)^{1/3} \text{ (ksi)} \leq 0.6 f_y$$

where  $d_c$  and  $A$  are as defined in AASHTO LRFD 5.7.3.4.

- b) For crack control the bar spacing cannot exceed the value from the following formula:

$$s \leq 700 * 0.75 / B_s f_s - 2 d_c$$

where  $B_s = 1 + d_c / .7(h - d_c)$

$d_c$  = concrete cover plus  $\frac{1}{2}$  bar diameter less  $\frac{1}{2}$  inch wearing surface (in)

$f_s$  = tensile stress in reinforcement at the service limit state (ksi)

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$h$  = slab depth minus ½ inch wearing surface (in)

Note: In the development of Table 17.1A and 17.2A the bar spacing crack control criteria was not satisfied if it required an excessive amount of additional steel above that required for strength. This can be justified because the arch action of the deck between girders is ignored and the bar stresses computed by beam action only are much larger than the actual stresses that occur for in place decks.

B. Longitudinal Reinforcement

The amount of bottom longitudinal reinforcement required is as stated in AASHTO LRFD 9.7.3.2. It is based on a percentage of the transverse moment positive steel. The percentage equals 220 divided by the square root of the effective span length in feet. The maximum required percentage is 67%. The minimum amount of longitudinal reinforcement required for temperature and shrinkage in the top layer and in the bottom layer is given by AASHTO Article LRFD 5.10.8.2.  $A_s = 0.11 * A_g / f_y$

$f_y = 60$        $A_g = 12 * T$        $A_s = 0.022 * T$  The other minimum amount of longitudinal steel in both layers used by WisDOT is #4 bars at 9" spacing to reduce transverse deck cracking. Identical amounts of steel are placed in both the top and bottom layer and is uniformly spaced from edge to edge of slab.

When continuous steel girders are not designed for negative composite action the AASHTO Code requires an area of steel in both the top and bottom layer equal to 1% of the cross-section area of the slab in the span negative moment regions. The "d" value used for this computation is the total slab depth minus the wearing surface. This steel is uniformly spaced from edge to edge of slab in the top and bottom layer and the top layer may be spaced differently than the top longitudinal steel in the span positive moment regions. Table 17.1B uses the same longitudinal bar spacings throughout a given bridge deck. AASHTO requires that two-thirds of this reinforcement be placed in the top layer. The epoxy coated reinforcement must be extended into the positive moment region beyond the shear connectors at least 46 bar diameters. Also, these bars should lap the number of shear connectors required to develop the yield strength in the bars.

C. Empirical Design of Slab on Girders

AASHTO LRFD 9.7.2 allows the empirical design method for slabs on girders. This method recognizes that an arching action occurs in the slab between the girders and that the true structural action is more complex than just pure bending alone.

WisDOT has tried several empirical deck designs beginning in the 1980's. When the performance of these decks is based on deck cracking they have underperformed the decks designed by the conventional method. In addition to the normal transverse deck cracks about a third of these decks have also developed longitudinal cracks between girders. The only advantage to using the empirical design method is the savings in cost due to a slight reduction in the pounds of transverse deck steel required.

In addition to the conditions in AASHTO LRFD that must be satisfied when using the empirical design method WisDOT has imposed the following additional conditions in an attempt to eliminate the longitudinal cracking.

For an 8" slab the maximum girder spacing is 7'-0.

For an 8.5" slab the maximum girder spacing is 8'-0.

For an 9'-0" slab the maximum girder spacing is 9'-0.

These limits are based on an area of steel of .28 inches/ft. (#4 @ 8.5) providing a load factor of 1.25 for dead load and 1.3 for live load. For empirical deck design use the following steel in both top and bottom of deck and the same bar clearance as shown in Figure 17.1. See Table 17.1A for continuity steel.

Transverse Reinforcing Steel      #4 @ 8.5

Longitudinal Reinforcing Steel      #4 @ 9.0

The empirical design method should not be used on bridge decks with high/heavy truck traffic. Also approval from WisDOT is required. The minimum slab overhang beyond centerline of exterior girder is 5 times the slab depth or 3 times the slab depth with a composite concrete parapet. The steel requirements stated in 17.3 (4) of this manual must also be supplied.

## MAXIMUM LRFD LIVE LOAD MOMENTS FOR DECK SLABS ON GIRDER, KIP-FT./FT.

GIRDER SPACING "S" FT-IN	POSITIVE MOMENT	NEGATIVE MOMENT					
		DISTANCE FROM CL OF GIRDER TO DESIGN SECTION FOR NEGATIVE MOMENT					
		3"	4"	5"	6"	10"	15"
4'-0	3.35	2.07	1.96	1.85	1.74	1.57	1.33
4'-3	3.50	2.25	2.15	2.05	1.95	1.60	1.36
4'-6	3.66	2.58	2.41	2.20	2.06	1.63	1.39
4'-9	3.83	2.88	2.68	2.47	2.30	1.81	1.49
5'-0	4.00	3.16	2.95	2.75	2.55	1.99	1.54
5'-3	4.14	3.42	3.21	2.99	2.78	2.17	1.61
5'-6	4.28	3.69	3.47	3.24	3.02	2.36	1.68
5'-9	4.38	3.93	3.69	3.45	3.22	2.53	1.75
6'-0	4.48	4.18	3.92	3.66	3.43	2.69	1.85
6'-3	4.62	4.35	4.09	3.83	3.59	2.82	1.93
6'-6	4.77	4.52	4.27	4.01	3.76	2.94	2.01
6'-9	4.85	4.71	4.46	4.20	3.96	3.06	2.11
7'-0	4.93	4.90	4.65	4.40	4.16	3.31	2.23
7'-3	5.05	5.08	4.85	4.62	4.39	3.44	2.30
7'-6	5.17	5.27	5.05	4.84	4.61	3.57	2.51
7'-9	5.25	5.47	5.26	5.00	4.71	3.69	2.75
8'-0	5.33	5.65	5.37	5.09	4.81	3.80	2.96
8'-3	5.41	5.74	5.46	5.18	4.90	3.88	3.13
8'-6	5.50	5.82	5.54	5.26	4.98	3.96	3.28
8'-9	5.54	5.90	5.62	5.34	5.06	4.04	3.41
9'-0	5.59	5.97	5.69	5.41	5.13	4.09	3.51
9'-3	5.67	6.03	5.75	5.47	5.19	4.21	3.64
9'-6	5.76	6.31	6.02	5.74	5.46	4.45	3.86
9'-9	5.87	6.65	6.36	6.08	5.80	4.70	4.05
10'-0	5.98	6.99	6.70	6.41	6.13	4.97	4.25
10'-3	6.14	7.32	7.03	6.74	6.45	5.29	4.50
10'-6	6.30	7.64	7.35	7.06	6.77	5.60	4.75
10'-9	6.47	7.95	7.66	7.37	7.08	5.90	5.00
11'-0	6.65	8.26	7.96	7.67	7.38	6.20	5.24
11'-3	6.79	8.55	8.25	7.96	7.67	6.49	5.47
11'-6	6.93	8.84	8.54	8.25	7.96	6.77	5.70
11'-9	7.11	9.12	8.82	8.53	8.24	7.06	5.93

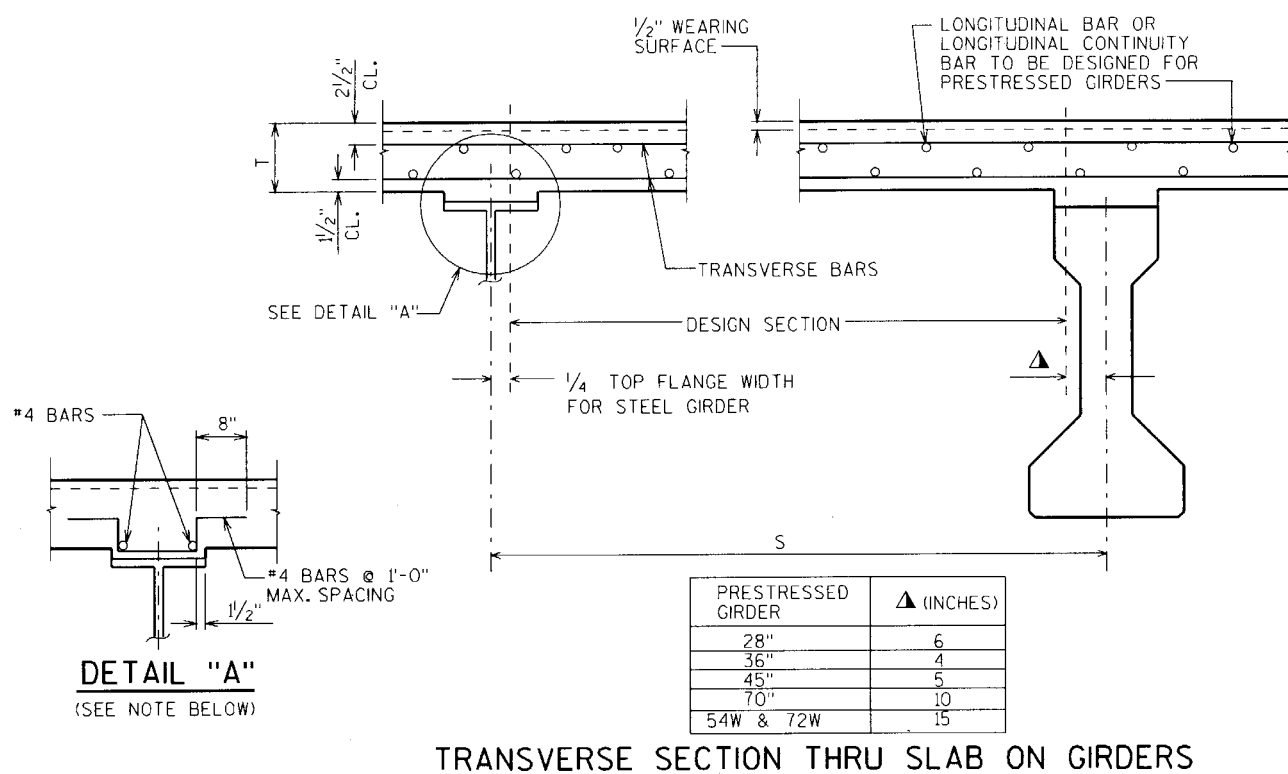
TABLE 17

## MAXIMUM LRFD LIVE LOAD MOMENTS FOR DECK SLABS ON GIRDER, KIP-FT./FT.

GIRDER SPACING "S" FT-IN	POSITIVE MOMENT	NEGATIVE MOMENT					
		DISTANCE FROM CL OF GIRDER TO DESIGN SECTION FOR NEGATIVE MOMENT					
		3"	4"	5"	6"	10"	15"
12'-0	7.30	9.40	9.10	8.80	8.51	7.33	6.15
12'-3	7.41	9.67	9.37	9.07	8.78	7.60	6.38
12'-6	7.53	9.93	9.63	9.33	9.04	7.86	6.62
12'-9	7.70	10.18	9.88	9.59	9.30	8.12	6.86
13'-0	7.87	10.43	10.13	9.84	9.55	8.37	7.08
13'-3	8.10	10.67	10.38	10.09	9.80	8.62	7.31
13'-6	8.34	10.91	10.61	10.32	10.03	8.86	7.53
13'-9	8.52	11.14	10.85	10.56	10.27	9.10	7.75
14'-0	8.71	11.37	11.08	10.79	10.50	9.34	7.97

Moments are based on 16 kip wheel loads distributed transversely over 27 inches. Multiple presence factors and the dynamic load allowance are included in the values. Longitudinal distribution is as stated in AASHTO LRFD 4.6.2.1.3. Moments are from one truck or two trucks four feet apart and a three girder system with no overhang or from AASHTO LRFD Table A4-1, whichever is less.

**TABLE 17 (CON'T)**

**DETAIL "A"** (SEE NOTE BELOW)

**NOTES:** Bottom transverse bar steel shall be supported by continuous bar chairs with a center to center spacing not to exceed 4' (1200 mm). One line of continuous bar chairs shall be placed near each edge of slab to support the ends of the bottom transverse bar steel.

Top longitudinal bar steel shall be supported by continuous bar chairs in transverse direction on 4 feet centers. For skews 20° and under place transverse bars along skew. For skews greater than 20° place transverse bars perpendicular to girders.

Use "Detail A" on deck replacements when shear connectors extend less than 2" into slab.

**FIGURE 17.1**

TRANSVERSE REINFORCING STEEL FOR DECK SLABS ON GIRDERS FOR  
NEW BRIDGES & DECK REPLACEMENTS, HL93 LOADING-LRFD

GIRDER SPACING "S"	SLAB THICKNESS "T"	DISTANCE FROM C/L OF GIRDER TO DESIGN SECTION (INCHES)					
		3"	4"	5"	6"	10"	15"
4'-6	8	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5
4'-9	8	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5	#4 @ 8.5
5'-0	8	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8
5'-3	8	#4 @ 7.5	#4 @ 7.5	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8
5'-6	8	#4 @ 7	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5
5'-9	8	#4 @ 7	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5
6'-0	8	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
6'-3	8	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
6'-6	8	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
6'-9	8	#4 @ 6	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7'-0	8	#5 @ 8.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7'-3	8	#5 @ 8.5	#5 @ 8.5	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
7'-6	8	#5 @ 8	#5 @ 8	#5 @ 8	#4 @ 6	#4 @ 6	#4 @ 6
7'-9	8	#5 @ 8	#5 @ 8	#5 @ 8	#4 @ 6	#4 @ 6	#4 @ 6
8'-0	8	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
8'-3	8.5	#5 @ 8	#5 @ 8.5	#5 @ 8.5	#4 @ 6	#4 @ 6	#4 @ 6
8'-6	8.5	#5 @ 8	#5 @ 8.5	#5 @ 8.5	#4 @ 6	#4 @ 6	#4 @ 6
8'-9	8.5	#5 @ 7.5	#5 @ 8	#5 @ 8.5	#4 @ 6	#4 @ 6	#4 @ 6
9'-0	8.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#4 @ 6	#4 @ 6
9'-3	8.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
9'-6	9	#5 @ 7.5	#5 @ 8	#5 @ 8.5	#4 @ 6	#4 @ 6	#4 @ 6
9'-9	9	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 8.5	#4 @ 6	#4 @ 6
10'-0	9	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
10'-3	9	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
10'-6	9	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8
10'-9	9.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
11'-0	9.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8
11'-3	9.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8
11'-6	9.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
11'-9	10	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8

**TABLE 17.1**

TRANSVERSE REINFORCING STEEL FOR DECK SLABS ON GIRDERS  
FOR NEW BRIDGES & DECK REPLACEMENTS, HL93 LOADING-LRFD

GIRDER SPACING "S"	TOTAL SLAB THICKNESS	DISTANCE FROM C/L OF GIRDER TO DESIGN SECTION (INCHES)					
Ft. – In.	In.	3"	4"	5"	6"	10"	15"
12'-0	10	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8
12'-3	10	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
12'-6	10	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
12'-9	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
13'-0	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
13'-3	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
13'-6	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
13'-9	11	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
14'-0	11	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5

Designed per AASHTO LRFD. Max. Allowable Design Stresses:  $f'_c = 4 \text{ ksi}$ ,  $f_y = 60 \text{ ksi}$ .

Top steel is 2 1/2" clear, bottom steel is 1 1/2" clear. Designed for 20 psf future wearing surface. Bars shown in table are for one layer only. Place identical steel in both top and bottom layer.

**TABLE 17.1 (CON'T)**

LONGITUDINAL REINFORCING STEEL FOR DECK SLABS ON GIRDERS FOR  
NEW BRIDGES AND DECK REPLACEMENTS, HL93 LOADING-LRFD

GIRDER SPACING "S" FT.-IN.	SLAB THICKNESS "T" INCHES	* BAR SIZE & SPACING INCHES	** STEEL GIRDER CONTINUITY BAR SIZE & SPACING (INCHES)	
			TOP LAYER	BOTTOM LAYER
4'-6	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
4'-9	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
5'-0	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
5'-3	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
5'-6	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
5'-9	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
6'-0	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
6'-3	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
6'-6	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
6'-9	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
7'-0	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
7'-3	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
7'-6	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
7'-9	8	#4 @ 8.5 or 9	#6 @ 8.5	#4 @ 8.5
8'-0	8	#4 @ 8.5	#6 @ 8.5	#4 @ 8.5
8'-3	8.5	#4 @ 8 or 9	#6 @ 8	#4 @ 8
8'-6	8.5	#4 @ 8 or 9	#6 @ 8	#4 @ 8
8'-9	8.5	#4 @ 8 or 9	#6 @ 8	#4 @ 8
9'-0	8.5	#4 @ 8 or 9	#6 @ 8	#4 @ 8
9'-3	8.5	#4 @ 8 or 8.5	#6 @ 8	#4 @ 8
9'-6	9	#4 @ 7.5 or 9	#6 @ 7.5	#4 @ 7.5
9'-9	9	#4 @ 7.5 or 8.5	#6 @ 7.5	#4 @ 7.5
10'-0	9	#4 @ 7.5 or 8.5	#6 @ 7.5	#4 @ 7.5
10'-3	9	#4 @ 7.5 or 8	#6 @ 7.5	#4 @ 7.5
10'-6	9	#4 @ 7.5 or 8	#6 @ 7.5	#4 @ 7.5
10'-9	9.5	#4 @ 7 or 8.5	#6 @ 7	#4 @ 7
11'-0	9.5	#4 @ 7 or 8	#6 @ 7	#4 @ 7
11'-3	9.5	#4 @ 7 or 8	#6 @ 7	#4 @ 7
11'-6	9.5	#4 @ 7 or 7.5	#6 @ 7	#4 @ 7
11'-9	10	#4 @ 7 or 8	#6 @ 7	#4 @ 7

**TABLE 17.1A**

LONGITUDINAL REINFORCING STEEL FOR DECK SLABS ON GIRDERS FOR  
NEW BRIDGES AND DECK REPLACEMENTS, HL93 LOADING-LRFD

GIRDER SPACING "S" FT.-IN.	SLAB THICKNESS "T" INCHES	* BAR SIZE & SPACING INCHES	** STEEL GIRDER CONTINUITY BAR SIZE & SPACING (INCHES)	
			TOP LAYER	BOTTOM LAYER
12'-0	10	#4 @ 7 or 8	#6 @ 7	#4 @ 7
12'-3	10	#4 @ 7 or 7.5	#6 @ 7	#4 @ 7
12'-6	10	#4 @ 7 or 7.5	#6 @ 7	#4 @ 7
12'-9	10.5	#4 @ 6.5 or 7.5	#6 @ 6.5	#4 @ 6.5
13'-0	10.5	#4 @ 6.5 or 7.5	#6 @ 6.5	#4 @ 6.5
13'-3	10.5	#4 @ 6.5 or 7.5	#6 @ 6.5	#4 @ 6.5
13'-6	10.5	#4 @ 6.5 or 7.5	#6 @ 6.5	#4 @ 6.5
13'-9	11	#4 @ 6 or 7.5	#6 @ 6	#4 @ 6
14'-0	11	#4 @ 6 or 7.5	#6 @ 6	#4 @ 6

Designed per AASHTO LRFD. Max. Allowable Design Stresses:  $f'_c = 4 \text{ ksi}$ ,  $f_y = 60 \text{ ksi}$ . Designed for 20 psf future wearing surface. Bars shown in table are for one layer only. Place identical steel in both top and bottom layer except for continuity steel.

\* Use smaller spacing when steel girder continuity steel is required.

\*\* Use for deck slabs on steel girders in negative moment region when not designed for negative moment composite action.

**TABLE 17.1A (CON'T.)**

TRANSVERSE REINFORCING STEEL FOR DECK SLABS ON GIRDERS FOR DECK  
REPLACEMENTS, HL93 LOADING

GIRDER SPACING S	SLAB THICKNESS	DISTANCE FROM C/L OF GIRDER TO DESIGN SECTION					
		3"	4"	5"	6"	10"	15"
4'-0	6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
4'-3	6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
4'-6	6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
4'-9	6.5	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
5'-0	6.5	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
5'-3	6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
5'-6	6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
5'-9	6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6'-0	6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6'-3	6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6'-6	6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6'-9	6.5	#5 @ 6	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
7'-0	6.5	#5 @ 6	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5
4'-0	7	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5
4'-3	7	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5
4'-6	7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
4'-9	7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
5'-0	7	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
5'-3	7	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
5'-6	7	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
5'-9	7	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
6'-0	7	#5 @ 8	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
6'-3	7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6'-6	7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6'-9	7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7'-0	7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7'-3	7	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
7'-6	7	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
7'-9	7	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
8'-0	7	#5 @ 6	#5 @ 6	#5 @ 6.5	#5 # 7	#5 @ 7	#5 @ 7
4'-0	7.5	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8
4'-3	7.5	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8	#4 @ 8
4'-6	7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5
4'-9	7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5
5'-0	7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5	#4 @ 7.5
5'-3	7.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
5'-6	7.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
5'-9	7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5

TABLE 17.2

TRANSVERSE REINFORCING STEEL FOR DECK SLABS ON GIRDERS FOR DECK  
REPLACEMENTS, HL93 LOADING

GIRDER SPACING S	SLAB THICKNESS	DISTANCE FROM C/L OF GIRDER TO DESIGN SECTION					
		3"	4"	5"	6"	10"	15"
6'-0	7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
6'-3	7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
6'-6	7.5	#5 @ 8.5	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
6'-9	7.5	#5 @ 8	#5 @ 8.5	#4 @ 6	#4 @ 6	#4 @ 6	#4 @ 6
7'-0	7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7'-3	7.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7'-6	7.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7'-9	7.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
8'-0	7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
8'-3	7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
8'-6	7.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
8'-9	7.5	#5 @ 6	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5
9'-0	7.5	#5 @ 6	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5
9'-3	7.5	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
9'-6	7.5	#5 @ 6	#5 @ 6	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5

Design per AASHTO LRFD. Max. Allowable Design Stresses:  $f'_c = 4 \text{ ksi}$ ,  $f_y = 60 \text{ ksi}$ . Top steel is 2 1/2" clear, bottom steel is 1 1/2" clear. Designed for 20 psf future wearing surface. Bars shown in table are for one layer only. Place identical steel in both top and bottom layer.

**TABLE 17.2 (CON'T)**

LONGITUDINAL REINFORCING STEEL FOR DECK SLABS ON GIRDERS FOR DECK  
REPLACEMENTS, HL93 LOADING

GIRDER SPACING "S" FT-IN	SLAB THICKNESS "T" INCHES	BAR SIZE & SPACING INCHES	** STEEL GIRDER CONTINUITY STEEL BAR SIZE & SPACING (INCHES)	
			TOP LAYER	BOTTOM LAYER
4'-0	6.5	#4 @ 9	#6 @ 9	#4 @ 9
4'-3	6.5	#4 @ 9	#6 @ 9	#4 @ 9
4'-6	6.5	#4 @ 9	#6 @ 9	#4 @ 9
4'-9	6.5	#4 @ 9	#6 @ 9	#4 @ 9
5'-0	6.5	#4 @ 9	#6 @ 9	#4 @ 9
5'-3	6.5	#4 @ 8	#6 @ 8	#4 @ 8
5'-6	6.5	#4 @ 8	#6 @ 8	#4 @ 8
5'-9	6.5	#4 @ 7.5	#5 @ 7.5	#4 @ 7.5
6'-0	6.5	#4 @ 7.5	#5 @ 7.5	#4 @ 7.5
6'-3	6.5	#4 @ 7	#5 @ 7	#4 @ 7
6'-6	6.5	#4 @ 7	#5 @ 7	#4 @ 7
6'-9	6.5	#4 @ 7	#5 @ 7	#4 @ 7
7'-0	6.5	#4 @ 7	#5 @ 7	#4 @ 7
4'-0	7	#4 @ 9	#6 @ 9	#4 @ 9
4'-3	7	#4 @ 9	#6 @ 9	#4 @ 9
4'-6	7	#4 @ 9	#6 @ 9	#4 @ 9
4'-9	7	#4 @ 9	#6 @ 9	#4 @ 9
5'-0	7	#4 @ 9	#6 @ 9	#4 @ 9
5'-3	7	#4 @ 9	#6 @ 9	#4 @ 9
5'-6	7	#4 @ 9	#6 @ 9	#4 @ 9
5'-9	7	#4 @ 9	#6 @ 9	#4 @ 9
6'-0	7	#4 @ 9	#6 @ 9	#4 @ 9
6'-3	7	#4 @ 8	#6 @ 8	#4 @ 8
6'-6	7	#4 @ 8	#6 @ 8	#4 @ 8
6'-9	7	#4 @ 7.5	#6 @ 7.5	#4 @ 7.5
7'-0	7	#4 @ 7.5	#6 @ 7.5	#4 @ 7.5
7'-3	7	#4 @ 7	#5 @ 7	#4 @ 7
7'-6	7	#4 @ 7	#5 @ 7	#4 @ 7
7'-9	7	#4 @ 7	#5 @ 7	#4 @ 7
8'-0	7	#4 @ 7	#5 @ 7	#4 @ 7
4'-0	7.5	#4 @ 9	#6 @ 9	#4 @ 9
4'-3	7.5	#4 @ 9	#6 @ 9	#4 @ 9
4'-6	7.5	#4 @ 9	#6 @ 9	#4 @ 9
4'-9	7.5	#4 @ 9	#6 @ 9	#4 @ 9

**TABLE 17.2A**

LONGITUDINAL REINFORCING STEEL FOR DECK SLABS ON GIRDERS FOR DECK  
REPLACEMENTS, HL93 LOADING

GIRDER SPACING "S" FT-IN	SLAB THICKNESS "T" INCHES	BAR SIZE & SPACING INCHES	** STEEL GIRDER CONTINUITY STEEL BAR SIZE & SPACING (INCHES)	
			TOP LAYER	BOTTOM LAYER
5'-0	7.5	#4 @ 9	#6 @ 9	#4 @ 9
5'-3	7.5	#4 @ 9	#6 @ 9	#4 @ 9
5'-6	7.5	#4 @ 9	#6 @ 9	#4 @ 9
5'-9	7.5	#4 @ 9	#6 @ 9	#4 @ 9
6'-0	7.5	#4 @ 9	#6 @ 9	#4 @ 9
6'-3	7.5	#4 @ 9	#6 @ 9	#4 @ 9
6'-6	7.5	#4 @ 9	#6 @ 9	#4 @ 9
6'-9	7.5	#4 @ 9	#6 @ 9	#4 @ 9
7'-0	7.5	#4 @ 8.5	#6 @ 8.5	#4 @ 8.5
7'-3	7.5	#4 @ 8.5	#6 @ 8.5	#4 @ 8.5
7'-6	7.5	#4 @ 8.5	#6 @ 8.5	#4 @ 8.5
7'-9	7.5	#4 @ 8.5	#6 @ 8.5	#4 @ 8.5
8'-0	7.5	#4 @ 7.5	#6 @ 7.5	#4 @ 7.5
8'-3	7.5	#4 @ 7.5	#6 @ 7.5	#4 @ 7.5
8'-6	7.5	#4 @ 7	#6 @ 7	#4 @ 7
8'-9	7.5	#4 @ 7	#6 @ 7	#4 @ 7
9'-0	7.5	#4 @ 7	#6 @ 7	#4 @ 7
9'-3	7.5	#4 @ 7	#6 @ 7	#4 @ 7
9'-6	7.5	#4 @ 7	#6 @ 7	#4 @ 7

Designed per AASHTO LRFD. Max. Allowable Design Stresses:  $f'_c = 4 \text{ ksi}$ ,  $f_y = 60 \text{ ksi}$ .

Designed for 20 psf future wearing surface. Bars shown in table are for one layer only. Place identical steel in both top and bottom layer except for continuity steel.

\*\* Use for deck slabs on steel girders in negative moment region when not designed for negative moment composite action.

**TABLE 17.2A (CON'T)**

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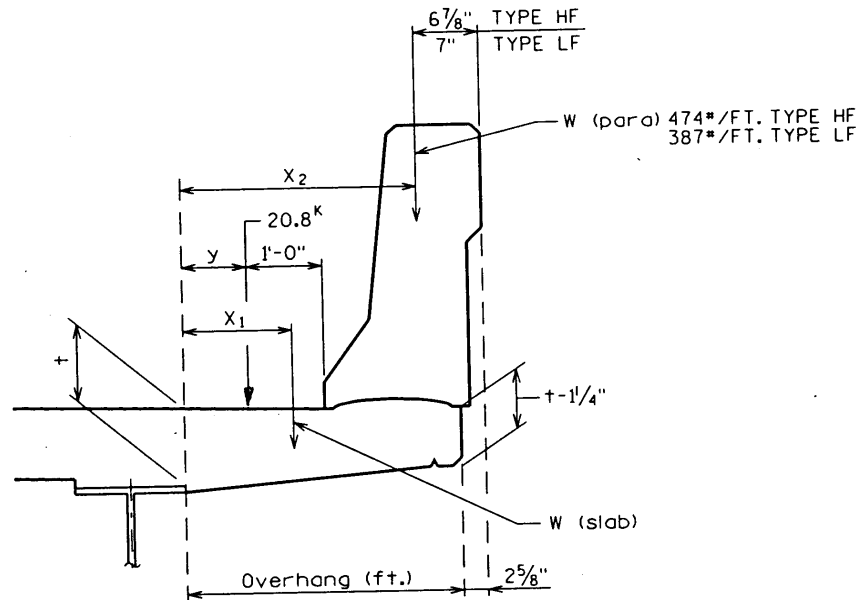
(4) Cantilever Slab Design

Current design practice limits the slab overhang length to 3'-7" measured from the centerline of the exterior girder to the edge of slab. The overhang length has been limited to prevent rotation of the girder and/or bending of the girder web during construction caused by the eccentric load from the cantilevered forming brackets. The upper portion of these brackets attach to the girder top flange and the lower portion bears against the girder web. If the girder rotates or the web bends at the bracket bearing point the end of the bracket will move downward because of bracket rotation. If the rails supporting the paving machine are located near the end of the bracket, the paving machine will move downward more than the girder and the anticipated profile grade line will not be achieved. Factors affecting girder rotation are diaphragm spacing, stiffness, and connections, and girder torsional stiffness. Stiffener spacing and web thickness will affect web bending. A 4'-6" overhang may be used where a curved roadway is placed on straight girders at the discretion of the designer.

A minimum area of slab reinforcement, independent of overhang length is required to carry the transverse railing loads specified in AASHTO. The minimum steel required at the face of railing is shown in Table 17.3 and 17.4 for "X" equal to 1 to 3 feet. Beyond 3 feet Truck Load governs. Additional reinforcing steel may be required at the end of the cantilever to distribute the transverse railing loads. The slab thickness "t", is the slab thickness at the face of girder flange if truck load governs or the slab thickness at the point under investigation if railing load governs. The tables are based on the controlling truck wheel load or rail load cases as follows:

A. Truck Load

The design moment is computed for slab and parapet dead load plus a live loading of a truck wheel including impact. Reference is made to AASHTO for live load wheel distribution. The effective length of distribution equals  $E_1 = 0.8y + 3.75$  in feet.  $y$  is the distance from point of load to the support in feet.



Moments are computed about the edge of the girder flange as follows:

$$M(\text{slab}) = W(\text{slab})X_1$$

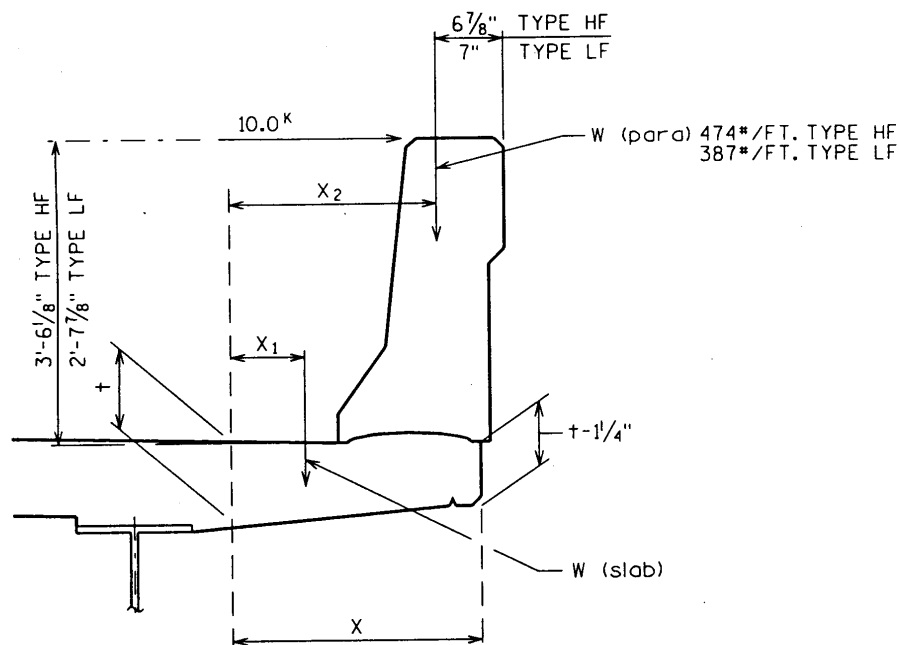
$$M(\text{para}) = W(\text{para})X_2$$

$$M(LL) = \frac{20.8y}{E_1}$$

The design moment for truck live load equals the summation of the above moments multiplied by the strength design load factors, 1.3 for dead load and 2.17 for live load.

#### B. Rail Load

The design moment is computed for slab and parapet dead loads plus a transverse rail live load of 10 kips placed at the top of the concrete parapet. For type "F" steel railing a 5 kip load is placed at the center of each rail. The same design moment is used for the type "W" steel railing as are used for the type "F". Reference is made to AASHTO 3.24.5.2 for live load rail distribution. If a parapet is employed the distribution length equals  $E_2 = 0.8x + 5.0$  in feet.  $X$  is the distance from the center of the parapet overlap or center of post to the point under investigation. If no parapet is used,  $E_2 = 0.8x + 3.75$ .



Moments are computed about the point under investigation.

$$M(\text{slab}) = W(\text{slab})X_1$$

$$M(\text{para}) = W(\text{para})X_2$$

$$M(LL) = \frac{10.0(34.8 + .5(t - 2.8))}{E_2}$$

The design moment for rail live load equals the summation of the above moments multiplied by the strength design load factors, 1.3 for dead load and 2.17 for live load.

The procedures outlined can be used for values not on Table 17.3 or 17.4 such as overhang lengths greater than five feet or different railing configurations.

For concrete type superstructures, the designer is required to consider the Rail Loading and provide adequate transverse bar steel area, accordingly. For "X" equal to 1.00 foot, minimum area of slab reinforcement for both concrete parapet and steel railing type "F" or "W" is as follows:

"t" (inches)	13	14	15	16	17	18	19	20
Reinforcing Steel (ln <sup>2</sup> /ft.)	.31	.29	.26	.25	.23	.22	.21	.20
"t" (mm)	325	350	375	400	425	450	475	500
Reinforcing Steel (mm <sup>2</sup> /m)	656	614	550	529	487	466	445	423

## REINFORCING STEEL FOR CANTILEVER DECK SLABS WITH SLOPE FACE PARAPET

$\frac{LF}{HS20}$   
LOADING IN<sup>2</sup>/FT

"X" OR OVERHANG (FT)	"t" (INCHES)							
	7.5	8	8.5	9	9.5	10	10.5	11
1.00	.689	.612	.553	.506	.467	.434	.406	.382
1.25	.671	.597	.539	.473	.455	.423	.396	.373
1.50	.655	.583	.527	.482	.446	.415	.388	.365
1.75	.642	.572	.517	.473	.437	.407	.381	.359
2.00	.630	.562	.507	.466	.430	.401	.375	.353
2.25	.621	.554	.501	.459	.425	.396	.371	.349
2.50	.613	.547	.496	.454	.420	.391	.367	.346
2.75	.606	.541	.491	.450	.417	.388	.364	.343
3.00	.601	.537	.487	.447	.414	.386	.362	.342
3.25	.637	.565	.509	.465	.428	.397	.370	.347
3.50	.768	.678	.609	.554	.509	.471	.439	.411
3.75	.897	.787	.704	.639	.586	.542	.504	.472
4.00	1.024	.893	.796	.720	.659	.609	.566	.530
4.25	1.151	.976	.884	.798	.729	.673	.625	.585
4.50		1.098	.970	.874	.797	.734	.682	.637
4.75		1.198	1.054	.947	.862	.794	.736	.687
5.00		1.297	1.137	1.018	.925	.851	.788	.736

Max. Allowable Design Stresses:  $f_c' = 4000$  psi,  $f_y = 60$  ksi, Top Steel 2 1/2" Clear. See pages 17, 18 and 19 for definition of overhang distance and "X". For "OVERHANG" greater than 3.00 feet Truck Load controls. Values in left column of 3.00 feet and less are "X" values.

TABLE 17.3a

Note: Multiply in<sup>2</sup>/ft. by 2117 to get mm<sup>2</sup>/m

REINFORCING STEEL FOR CANTILEVER DECK SLABS WITH SLOPE FACE PARAPET "HF",  
HS20 LOADING IN<sup>2</sup>/FT

"X" OR OVERHANG (FT)	"t" (INCHES)							
	7.5	8	8.5	9	9.5	10	10.5	11
1.00	.915	.803	.720	.654	.601	.556	.519	.487
1.25	.888	.781	.700	.637	.585	.542	.506	.475
1.50	.865	.762	.684	.622	.572	.530	.495	.464
1.75	.845	.745	.669	.609	.560	.520	.485	.455
2.00	.828	.731	.657	.598	.550	.510	.477	.448
2.25	.813	.718	.646	.587	.542	.503	.470	.441
2.50	.801	.708	.637	.581	.535	.497	.464	.436
2.75	.790	.699	.629	.574	.529	.491	.459	.432
3.00	.782	.692	.623	.569	.525	.487	.456	.429
3.25	.775	.686	.619	.565	.521	.484	.453	.426
3.50	.790	.697	.625	.568	.522	.483	.451	.425
3.75	.922	.808	.722	.655	.600	.555	.516	.484
4.00	1.053	.916	.816	.738	.675	.623	.580	.542
4.25	1.184	1.023	.907	.818	.747	.687	.640	.598
4.50		1.127	.995	.895	.816	.751	.697	.652
4.75		1.23	1.081	.970	.883	.812	.753	.703
5.00			1.166	1.043	.947	.870	.806	.752

Max. Allowable Design Stresses:  $f_c' = 4000$  psi,  $f_y = 60$  ksi, Top Steel 2 1/2" Clear. See pages 17, 18 and 19 for definition of overhang distance and "X". For "OVERHANG" greater than 3.25 feet Truck Load controls. Values in left column of 3.25 feet and less are "X" values.

**TABLE 17.3b**

Note: Multiply in<sup>2</sup>/ft. by 2117 to get mm<sup>2</sup>/m

REINFORCING STEEL FOR CANTILEVER DECK SLABS WITH CONCRETE PARAPET B  
HS20 LOADING IN<sup>2</sup>/FT

"X" OR OVERHANG (FT)	"t" (INCHES)							
	7.5	8	8.5	9	9.5	10	10.5	11
1.00	.686	.610	.551	.503	.465	.432	.404	.380
1.25	.667	.594	.537	.491	.453	.421	.394	.371
1.50	.651	.580	.524	.480	.443	.412	.386	.363
1.75	.637	.568	.514	.470	.435	.404	.379	.357
2.00	.625	.557	.505	.462	.427	.398	.373	.351
2.25	.615	.549	.497	.456	.421	.392	.368	.346
2.50	.607	.542	.491	.450	.416	.388	.364	.343
2.75	.599	.536	.486	.446	.412	.384	.360	.340
3.00	.594	.531	.482	.442	.409	.382	.358	.338
3.25	.702	.621	.559	.509	.468	.433	.404	.379
3.50	.830	.730	.654	.594	.545	.505	.470	.440
3.75	.956	.836	.746	.676	.619	.572	.532	.498
4.00	1.080	.938	.835	.754	.690	.637	.592	.553
4.25		1.039	.920	.830	.757	.698	.649	.606
4.50		1.137	1.004	.902	.823	.757	.703	.657
4.75		1.235	1.085	.973	.885	.814	.755	.705
5.00			1.164	1.042	.946	.869	.805	.751

Max. Allowable Design Stresses:  $f_c' = 4000$  psi,  $f_y = 60$  ksi, Top Steel 2 1/2" Clear. See pages 17, 18 and 19 for definition of overhang distance and "X". For "OVERHANG" greater than 3.00 feet Truck Load controls. Values in left column of 3.00 feet and less are "X" values.

**TABLE 17.3c**

Note: Multiply in<sup>2</sup>/ft. by 2117 to get mm<sup>2</sup>/m

REINFORCING STEEL FOR CANTILEVER DECK SLABS WITH STEEL RAILING,  
TYPE "F" OR "W", HS20 LOADING, IN<sup>2</sup>/FT

"X" OR OVERHANG (FT)	"t" (INCHES)							
	7.5	8	8.5	9	9.5	10	10.5	11
1.00	.649	.579	.524	.481	.445	.415	.389	.367
1.25	.618	.552	.501	.459	.425	.397	.372	.351
1.50	.591	.529	.480	.441	.408	.381	.358	.337
1.75	.567	.508	.461	.424	.393	.367	.345	.325
2.00	.545	.489	.445	.409	.380	.355	.333	.315
2.25	.527	.473	.430	.396	.368	.344	.323	.305
2.50	.510	.458	.418	.385	.357	.334	.314	.297
2.75	.495	.445	.406	.374	.348	.326	.307	.290
3.00	.540	.481	.435	.397	.366	.340	.317	.298
3.25	.659	.585	.526	.480	.442	.409	.382	.358
3.50	.775	.684	.614	.558	.513	.475	.443	.415
3.75	.888	.779	.697	.633	.580	.537	.500	.468
4.00	.999	.871	.777	.704	.645	.596	.554	.519
4.25	1.108	.961	.854	.772	.706	.652	.606	.567
4.50		1.048	.929	.837	.765	.705	.655	.613
4.75		1.333	1.001	.900	.821	.756	.702	.656
5.00		1.217	1.071	.961	.876	.806	.748	.698

Max. Allowable Design Stresses:  $f_c' = 4000$  psi,  $f_y = 60$  ksi, Top Steel 2 1/2" Clear. See pages 17, 18 and 19 for definition of overhang distance and "X". For "OVERHANG" greater than 2.75 feet Truck Load controls. Values in left column of 2.75 feet and less are "X" values.

TABLE 17.4

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## 17.4 CONSTRUCTION JOINTS

Optional transverse construction joints are permitted on continuous concrete deck structures to limit the concrete volume in a single pour. Refer to Standard 24.11 for optimum slab pouring sequence. On structures over 300 feet (90 meters) long, transverse construction joints, if used are to be placed at the 0.6 of the span length beyond the pier in the direction of pour. For continuous prestressed girder bridges, locate the optional transverse construction joints midway between the cut-off points for continuity reinforcing steel or at the 0.75 of the span, whichever is closest to the pier.

The rate of placing concrete for continuous steel girders shall equal or exceed 0.5 span length per hour but need not exceed 100 yd<sup>3</sup> (75 m<sup>3</sup>) per hour. Transverse construction joints may be omitted with Approval of Bureau of Structures.

Optional longitudinal construction joints, if used, are to be approved by the engineer and preferably located beneath the median or parapet. When the width of a superstructure exceeds 90 feet a longitudinal construction joint with reinforcement thru the joint shall be detailed. Locate longitudinal joints beneath a parapet or median if possible. Otherwise locate joint along edge of lane line. Longitudinal joints should also be at least 6 inches from edge of top flange of girder. Open joints may be used if in a median or between parapets. Consideration should be given to sealing open joints with compression seals or other sealants.

The structure plans permit the contractor to propose an alternate construction joint schedule subject to approval of the engineer.

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## 17.5 BRIDGE DECK PROTECTIVE SYSTEMS

FHWA is encouraging states requiring the use of de-icers to employ bridge deck protective systems. The major problem of bridge deck deterioration is delamination of the concrete near the top mat of reinforcing steel followed by subsequent spalling of the surface concrete. Research indicates that the most prevalent cause of extensive deck deterioration is corrosion of the reinforcing steel due to the intrusion of chlorides into the concrete from repeated de-icer applications during snow and/or ice removal.

Currently, several types of bridge deck protective systems are available; some have been given FHWA approval based on their initial performance. Some of the more common types of protective systems are waterproofing membranes, epoxy coated, galvanized, or stainless steel clad reinforcing steel, microsilica modified concrete or polymer impregnated concrete, cathodic protection, and deck sealers. Epoxy coated rebars are preferred by WisDOT.

Structures other than box culverts that are designed to carry an earth fill, are required to have waterproofing membrane systems on the deck to protect the slab. This occurs on bridges designed for future grade changes.

Where an open railing allows drainage off the edge of the deck (slab), "Protective Surface Treatment" shall be applied to the side of the deck (slab) and along the bottom edge of deck (slab) for 3".

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**17.6 BRIDGE APPROACHES**

| The structure approach slab/pavement is part of the roadway design plans. Structure approach standards are provided in the Facilities Development Manual (FDM).

| The guidance for the selection of pavement types for bridge approaches is as shown in FDM  
| Procedure 14-10-15.

Considerations for site materials, drainage and backfill are provided in Chapter 12 – Abutments. Most approach pavement failures are related to the settlement of embankment or foundation materials. Past experience shows that significant settlement is most likely to occur where marginal materials are used. Designers are encouraged to provide perforated underdrains wrapped in geotextile fabric placed in a trench filled with crushed stone. Also, abutment backfill material should be granular in nature and consolidated under optimum moisture conditions.

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17.7 DESIGN OF PRECAST PRESTRESSED CONCRETE DECK PANELS

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## (1) General

An advantage of stay-in-place forms is that they can be placed in less time than it takes to place the forms for a conventional deck. There is also a labor savings because the extra step of removing deck forms is not required. Stay-in-place forms are often the preferred system for shallow box girders because of the difficulty of removing forms in a confined space.

If not detailed in the contract documents, precast concrete deck panels may be used at the option of the contractor provided the specifications permit their use. When a contractor elects to use precast deck panels at their option, then the contractor is responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the poured in place portion of the deck. Payment to a contractor who opts to use stay-in-place forms is based on the contract prices bid for the conventional cast-in-place deck.

Deck panels are only used between the inside faces of the exterior girders. The overhangs outside of the exterior girders are formed and the concrete placed in the same way as in a conventional poured-in-place deck. On skewed decks, the contractor may form and cast the skewed portion of the deck full depth or they may use skewed end deck panels which may be individually precast or saw-cut from square end planks.

A problem with decks formed with concrete deck panels is that cracks often form in the poured in place concrete over the transverse joints between panels. Reflection cracking is less of a problem when these panels are used on prestressed concrete girders than on steel girders. Simple span prestressed girder bridges will have less reflective cracking than continuous span prestressed girder bridges.

## (2) Deck Panel Design

The designs of precast prestressed concrete deck panels shown in Table 17.5 is based on AASHTO LRFD design criteria. Panels are designed for the HL93 design truck live load moment, dead load of the plastic concrete supported by the panels, a construction load of 50 pounds/sq. ft., dead load of the panels, and a future wearing surface of 20 pounds/sq. ft. The live load moments were obtained from a continuous beam analysis using three supports and either one or two trucks with two trucks being four feet apart. Wheel loads were distributed over the tire width. Load factor for live load is 1.75, and load factor for dead load is 1.25. A multiple presence factor of 1.2 was used for one truck. Impact is 33%. The live load moments obtained from this analysis were slightly smaller than those given in Table A4.1-1 in AASHTO.

At the request of the Precast plants only two strand sizes were used, 3/8 and 1/2 inch. They did not want the additional overhead of stocking 7/16 inch strand. Also strand spacing as per their request is in multiples of 2 inches. A 3 inch minimum

panel thickness was used even though AASHTO 9.7.4.3.1 requires a minimum thickness of 3.5 inches. The decision to use a 3 inch minimum panel was based on the successful use of 3 inch panels by other agencies over many years and that a minimum of 5 inches of poured-in-place concrete is preferred for crack control and bar steel placement. A 3.5 inch panel thickness would require a 8.5 inch deck which would not allow direct substitution of panels for a traditionally designed 8 inch deck. Also a study performed at Iowa State University determined that a 3 inch thick panel with coated 3/8 inch strands at midthickness and spaced at 6 inches along with epoxy-coated 6 x 6 – D6 x D6 welded wire fabric was adequate to prevent concrete splitting during strand detensioning. #3 bars placed perpendicular to the strands at 9" spacing also prevents concrete splitting. Panel thicknesses were increased by 1/2 inch whenever a strand spacing of less than 6 inches was required. 1/2 Inch strands were used in panels 4 inches thick or greater when 3/8 inch strands at 6 inch spacing were not sufficient.

The allowable tension stress in the panels equals  $6\sqrt{f'_c}$ . Transfer length of the strands is assumed to be 60 strand diameters at a stress of 202.5 ksi. The development length of the strands is assumed to be:

$$l_d = k (f_{ps} - 2/3 f_{pe}) d_b$$

where k is set equal to 1.6 based on proposed AASHTO specifications.

$d_b$  = Nominal strand diameter, inches

$f_{ps}$  = Average stress in prestressing steel at the time when the nominal resistance of the member is required, ksi.

$f_{pe}$  = Effective stress in prestressing steel after losses, ksi.

$l_d$  = Development length beyond critical section, inches.

Minimum panel width equals two times transfer length or two times development length whichever is greater.

The designs in Table 17.5 are based on uncoated prestressing strands. Grit-impregnated, epoxy-coated strands cost four times as much as uncoated strands but have about 1/2 the transfer and development length as uncoated strands. A cover of 1 1/4 inches is adequate to provide protection from chlorides for uncoated strands using a 5000 psi concrete mix. However for bridges with high traffic volume a 6000 psi mix is recommended.

AASHTO 9.7.4.3.2 does not require that the strands extend beyond the panels into the poured-in-place concrete over the girders. This simplifies construction of the

panels at the plant since they can be saw cut to the required length. Installation in the field is also simplified because extended strands may interfere with girder shear connectors. As a substitute for the strands that don't extend out of the panels, #4 bars spaced at twice the spacing of the transverse bars are placed on top of the panels over the girders in the poured in place concrete. These bars anchor the panels together to prevent or reduce longitudinal cracking over the ends of the panels and also resist any positive continuity moments that may develop. Also by not extending the strands into the poured-in-place concrete the uncoated strands are not exposed to chlorides that may seep through cracks that may develop in the poured-in-place concrete.

AASHTO 5.7.3.3.2 requires that the moment capacity of a flexural member be greater than 1.2 times the cracking moment based on the modulus of rupture. This requirement may be waived if the moment capacity is greater than 1.33 times the factored design moment. The purpose of this requirement is to provide a minimum amount of reinforcement in a flexural member so that a flexure failure will not be sudden or occur without warning. Tests have shown that for slabs on girders the failure mode is a punching shear failure and not a flexural failure. ACI 10.5.4 also recognizes the difference between slabs and beams and does not require the same minimum reinforcement for slabs. For these reasons AASHTO 5.7.3.3.2 was not considered in the designs of the panels shown in Table 17.5 although panels with a width of 6 feet or more do meet the requirements of AASHTO 5.7.3.3.2.

### (3) Reinforcing Steel for Poured in Place Concrete on Deck Panels

#### A. Transverse Reinforcement

The design of the transverse reinforcing steel in the poured in place concrete placed on deck panels is based on AASHTO LRFD design. The live load moments used to determine the size and spacing of the transverse bars placed in the top of the poured in place concrete are from TABLE A4.1-1 in AASHTO except that for "S" values less than 8'-0 the moments were obtained from a continuous beam analysis. The continuous beam analysis is based on three supports and either one or two trucks. Two trucks were placed four feet apart. For "S" less than 7', one truck gives larger moments than two. Wheel loads were distributed over the tire width. The load factor used for live load is 1.75 and 1.25 is used for dead load. A multiple presence factor of 1.2 is used for one truck. Impact is 33%. The continuous beam analysis for "S" values less than 8'-0 gave moments that are slightly smaller than those in TABLE A4.1-1 of AASHTO.

The reinforcing steel in the poured-in-place concrete is also designed for a future wearing surface of 20 pounds/sq. ft. With stay-in-place forms there are no negative moments from the dead load of the poured in place concrete.

The required reinforcing steel shown in Table 17.6 is based on both the strength requirement and crack control requirement. For crack control the

steel stress from service moments cannot exceed the value from the following formula:

$$f_s = 130 / (d_c A)^{1/3} \text{ (ksi)} \leq 0.6 f_y$$

where  $d_c$  = equals the thickness of concrete cover less 1/2 inch wearing surface measured from extreme tension fiber to center of the closest bar in inches, but a clear distance not greater than 2 inches for calculation purposes.

A = equals effective tension area in square inches of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars.

A concrete strength of 4000 psi was assumed and the haunch height over the girders was not considered. The "Distance from C/L of Girder to Design Section" is from AASHTO LRFD 4.6.2.1.6.

For prestress girders use the values in the following table.

GIRDER DEPTH (IN.)	DISTANCE (IN.)
28	6
36	4
45	5
54W	15
70	10
72W	15
82W	15

The reinforcing steel in Table 17.6 does not account for deck overhangs. Check Table 17.3 or 17.4 to see the minimum amount of steel required in the overhangs. Also for any portion of a deck not supported by deck panels use Table 17.1 for determining the required reinforcing steel.

#### B. Longitudinal Reinforcement

The longitudinal reinforcing steel, #4 @ 9" spacing, is from the AASHTO Standard Specifications, Article 9.18.2.4. For continuous prestress girders the longitudinal steel over the piers is the same as that required for a conventional deck. For steel girders see 17.3(3)A of this manual for longitudinal continuity steel.

#### (4) Details

Precast deck panels should extend a minimum of 1.5 inches beyond the face of

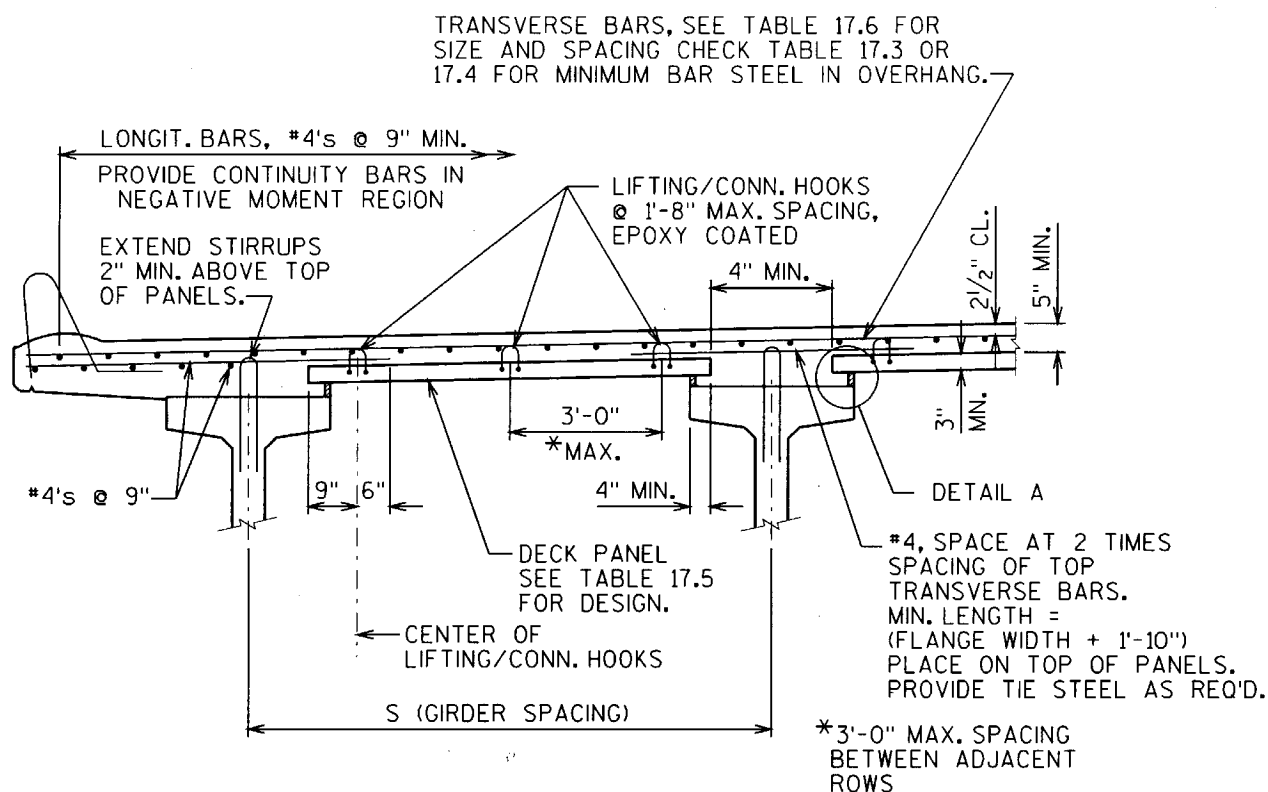
concrete diaphragms. The transverse joints between panels in adjacent bays should be staggered, preferably a distance about 1/2 panel length. Staggering the joints helps to minimize transverse reflective cracking.

Panels should never rest directly on a girder flange. According to AASHTO 9.7.4.3.4, "The ends of the panels shall be supported on a continuous mortar bed or shall be supported during construction in such a manner that the cast-in-place concrete flows into the space between the panel and the supporting component to form a concrete bedding". The minimum width of bearing on the flange of a girder for both concrete and mortar or grout support is 3 inches. See Figure 17.2 for additional information.

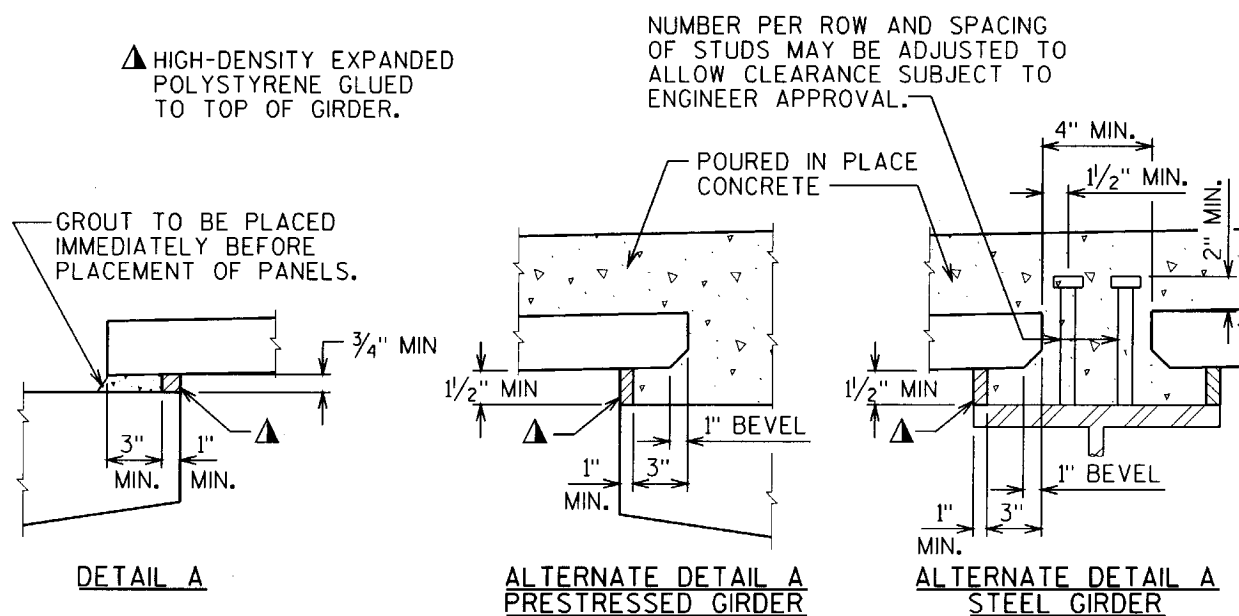
High-Density Expanded Polystyrene is used to support the panels prior to the placement of the poured in place concrete under the panel. The polystyrene is cut to the required haunch height so a constant slab thickness is maintained. High-Density Expanded Polystyrene is available in different strengths and it is the responsibility of the contractor to determine the strength required based on the vertical load that must be supported. Fiber board or sheathing panel supports are not allowed because the slight deflection of polystyrene compresses the concrete underneath the panel and results in less reflective longitudinal cracking along the panel edge.

When panels are supported on grout the main function of the polystyrene is to form the haunch height and to form a dam for the grout placement. The grout must be placed immediately before placement of panels and it is important that enough grout be placed so that the vertical load from the panels are supported by the grout and not by the polystyrene.

Some agencies specify a maximum haunch height and when it is exceeded allow the contractor to thicken the slab. Wisconsin does not specify a maximum haunch height and leaves that decision to the designer who is better informed to make that decision based on the specific situation of their project.



TRANSVERSE SECTION THRU SLAB ON GIRDERS WITH DECK PANELS



**FIGURE 17.2**

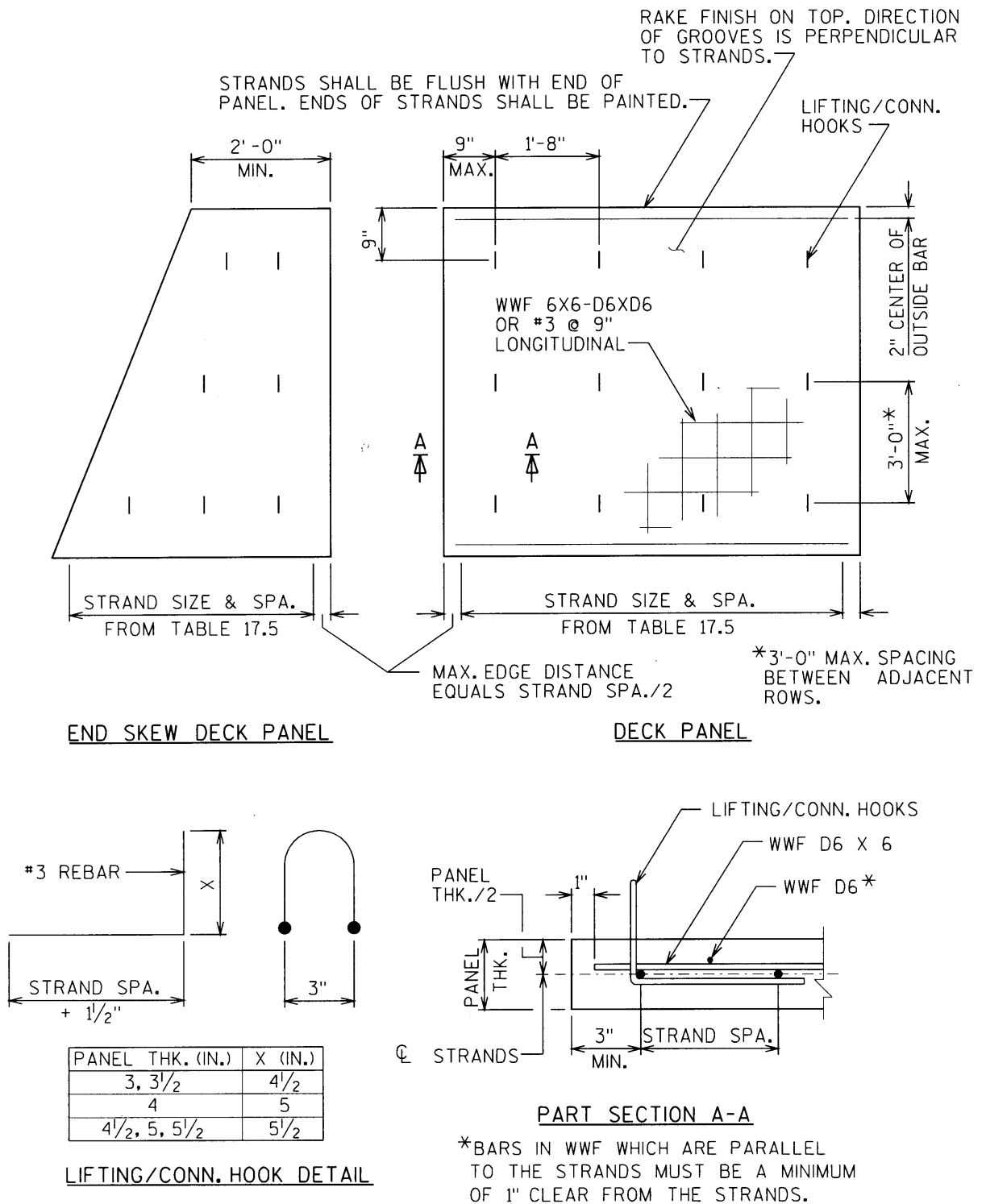


FIGURE 17.3

PRECAST PRESTRESSED CONCRETE DECK PANEL DESIGN TABLE  
 HL93 LOADING,  $f'_c = 5000$  psi,  $f'_c$  slab = 4000 psi,  $f_s = 270000$  psi (low-relax)

GIRDER SPACING "S" FT.-IN.	PANEL THICK. IN.	TOTAL SLAB THICK. IN.	TOP FLANGE WIDTH (INCHES)												12			16			18			24			30			48		
			STRAND			STRAND			STRAND			STRAND			STRAND			STRAND			STRAND			STRAND			STRAND			STRAND		
			Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$	Size	Spa.	$P_i$
			In.	In.	Kips	In.	In.	Kips	In.	In.	Kips	In.	In.	Kips	In.	In.	Kips	In.	In.	Kips	In.	In.	Kips	In.	In.	Kips	In.	In.	Kips	In.	In.	Kips
4'-6 <sup>(1)</sup>	3	8	3/8	10	11.48	3/8	10	*16.07	3/8	10	*16.07	3/8	10	13.77	3/8	10	13.77	3/8	10	*13.77												
4'-9	3	8	3/8	10	11.48	3/8	10	11.48	3/8	10	*16.07	3/8	10	13.77	3/8	10	13.77	3/8	10	*13.77												
5'-0	3	8	3/8	10	11.48	3/8	10	11.48	3/8	10	12.62	3/8	10	*16.07	3/8	10	*16.07	3/8	10	*13.77												
5'-3 <sup>(2)</sup>	3	8	3/8	10	11.48	3/8	10	12.62	3/8	10	12.62	3/8	10	12.62	3/8	10	*16.07	3/8	10	*14.92												
5'-6 <sup>(2)</sup>	3	8	3/8	10	11.48	3/8	10	12.62	3/8	10	12.62	3/8	10	13.77	3/8	10	13.77	3/8	10	*16.07												
5'-9 <sup>(2)</sup>	3	8	3/8	10	11.48	3/8	10	12.62	3/8	10	12.62	3/8	10	13.77	3/8	10	13.77	3/8	10	*16.07												
6'-0 <sup>(2)</sup>	3	8	3/8	10	12.62	3/8	10	12.62	3/8	10	12.62	3/8	10	13.77	3/8	10	13.77	3/8	10	14.92												
6'-3	3	8	3/8	10	12.62	3/8	10	12.62	3/8	10	13.77	3/8	10	13.77	3/8	10	13.77	3/8	10	14.92												
6'-6	3	8	3/8	10	13.77	3/8	10	13.77	3/8	10	13.77	3/8	10	14.92	3/8	10	14.92	3/8	10	16.07												
6'-9	3	8	3/8	10	14.92	3/8	10	13.77	3/8	10	13.77	3/8	10	14.92	3/8	10	14.92	3/8	10	16.07												
7'-0	3	8	3/8	10	16.07	3/8	10	14.92	3/8	10	13.77	3/8	10	14.92	3/8	10	14.92	3/8	10	16.07												
7'-3	3	8	3/8	10	17.21	3/8	10	16.07	3/8	10	14.92	3/8	10	14.92	3/8	10	14.92	3/8	10	16.07												
7'-6	3	8	3/8	8	14.92	3/8	10	17.21	3/8	10	16.07	3/8	10	16.07	3/8	10	14.92	3/8	10	16.07												
7'-9	3	8	3/8	8	16.07	3/8	8	14.92	3/8	10	17.21	3/8	10	17.21	3/8	10	16.07	3/8	10	16.07												
8'-0	3	8	3/8	8	17.21	3/8	8	16.07	3/8	8	14.92	3/8	8	14.92	3/8	8	16.07	3/8	8	16.07												
8'-3	3.5	8.5	3/8	8	14.92	3/8	10	17.21	3/8	10	17.21	3/8	10	17.21	3/8	10	14.92	3/8	10	14.92												
8'-6	3.5	8.5	3/8	8	16.07	3/8	8	14.92	3/8	8	14.92	3/8	8	14.92	3/8	8	16.07	3/8	8	16.07												
8'-9	3.5	8.5	3/8	8	17.21	3/8	8	16.07	3/8	8	16.07	3/8	8	16.07	3/8	8	17.21	3/8	8	17.21												
9'-0	3.5	8.5	3/8	6	13.77	3/8	8	17.21	3/8	8	17.21	3/8	8	17.21	3/8	8	14.92	3/8	8	14.92												
9'-3	3.5	8.5	3/8	6	14.92	3/8	6	13.77	3/8	6	13.77	3/8	6	13.77	3/8	6	16.07	3/8	6	16.07												
9'-6	3.5	9	3/8	6	14.92	3/8	6	13.77	3/8	6	13.77	3/8	6	13.77	3/8	6	16.07	3/8	6	16.07												
9'-9	3.5	9	3/8	6	16.07	3/8	6	14.92	3/8	6	14.92	3/8	6	14.92	3/8	6	17.21	3/8	6	17.21												
10'-0	3.5	9	3/8	6	17.21	3/8	6	16.07	3/8	6	16.07	3/8	6	16.07	3/8	6	17.21	3/8	6	17.21												
10'-3	4	9	1/2	10	26.87	1/2	12	31.00	1/2	12	31.00	1/2	12	31.00	1/2	12	26.87	1/2	12	26.87												
10'-6	4	9	1/2	10	28.94	1/2	10	28.94	1/2	10	28.94	1/2	10	28.94	1/2	10	26.87	1/2	12	26.87												
10'-9	4	9.5	1/2	10	31.00	1/2	10	28.94	1/2	10	28.94	1/2	10	28.94	1/2	10	26.87	1/2	12	26.87												
11'-0	4	9.5	1/2	10	31.00	1/2	10	31.00	1/2	10	31.00	1/2	10	31.00	1/2	10	28.94	1/2	12	28.94												
11'-3	4	9.5	1/2	8	26.87	1/2	8	26.87	1/2	8	26.87	1/2	8	26.87	1/2	8	26.87	1/2	10	26.87												
11'-6	4	9.5	1/2	8	28.94	1/2	8	28.94	1/2	8	28.94	1/2	8	28.94	1/2	8	26.87	1/2	10	26.87												
11'-9	4	10	1/2	8	31.00	1/2	8	31.00	1/2	8	31.00	1/2	8	31.00	1/2	8	28.94	1/2	10	28.94												

PRECAST PRESTRESSED CONCRETE DECK PANEL DESIGN TABLE  
 HL93 LOADING,  $f_c = 5000$  psi,  $f_s = 4000$  psi,  $f_s = 270000$  psi (low-relax)

GIRDER SPACING "S" FT.-IN.	PANEL THICK. IN.	TOTAL SLAB THICK. IN.	TOP FLANGE WIDTH (INCHES)											
			12			16			18			24		
			STRAND			STRAND			STRAND			STRAND		
			Size In.	Spa. In.	$P_i$ Kips	Size In.	Spa. In.	$P_i$ Kips	Size In.	Spa. In.	$P_i$ Kips	Size In.	Spa. In.	$P_i$ Kips
12'-0	4	10	1/2	8	31.00	1/2	8	28.94	1/2	8	26.87	1/2	10	31.00
12'-3	4.5	10	1/2	8	31.00	1/2	8	28.94	1/2	8	26.87	1/2	10	31.00
12'-6	4.5	10	1/2	8	31.00	1/2	8	28.94	1/2	8	26.87	1/2	10	31.00
12'-9	5	10.5	1/2	8	28.94	1/2	8	28.94	1/2	8	26.87	1/2	10	28.94
13'-0	5	10.5	1/2	8	31.00	1/2	8	28.94	1/2	8	26.87	1/2	10	28.94
13'-3	5.5	10.5	1/2	8	31.00	1/2	8	28.94	1/2	8	26.87	1/2	10	28.94
13'-6	5.5	10.5	1/2	6	24.80	1/2	8	31.00	1/2	8	28.94	1/2	8	26.87
13'-9	5.5	10.5	1/2	6	26.87	1/2	6	24.80	1/2	6	24.80	1/2	8	28.94
13'-9	5.5	11	1/2	6	24.80	1/2	8	31.00	1/2	8	28.94	1/2	8	26.87
14'-0	5.5	11	1/2	6	26.87	1/2	6	24.80	1/2	6	24.80	1/2	8	28.94

Designed per AASHTO LRFD specifications. Design loading includes 20 lbs. per square foot for future wearing surface and 50 psf construction load.  $P_i$ 's in Table are minimum required and may be increased to a maximum of  $.75 \times f_s \times A_s$  except for  $P_i$  values preceded by an asterisk. Strands are located at the centroid of the panels.

- (1) 30 inch flange width requires minimum panel width of 36 inches.
- (2) 48 inch flange width requires minimum panel width of 36 inches.

**TABLE 17.5**

TRANSVERSE REINFORCING STEEL FOR DECK SLABS ON PRECAST  
CONCRETE DECK PANELS, HL93 LOADING

GIRDER SPACING "S"	TOTAL SLAB THICKNESS in.	DISTANCE FROM C/L OF GIRDER TO DESIGN SECTION (INCHES)							
		3"	4"	5"	6"	10"	15"		
4'-6	8	#4 @ 10	#4 @ 11	#4 @ 11.5	#4 @ 12	#4 @ 12	#4 @ 12		
4'-9	8	#4 @ 9.5	#4 @ 10	#4 @ 10.5	#4 @ 11	#4 @ 12	#4 @ 12		
5'-0	8	#4 @ 9	#4 @ 9	#4 @ 9.5	#4 @ 10.5	#4 @ 12	#4 @ 12		
5'-3	8	#4 @ 8	#4 @ 8.5	#4 @ 9	#4 @ 9.5	#4 @ 11.5	#4 @ 12		
5'-6	8	#4 @ 7.5	#4 @ 8	#4 @ 8.5	#4 @ 9	#4 @ 11	#4 @ 12		
5'-9	8	#4 @ 7.5	#4 @ 7.5	#4 @ 8	#4 @ 8.5	#4 @ 10.5	#4 @ 12		
6'-0	8	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 8	#4 @ 10	#4 @ 12		
6'-3	8	#4 @ 7	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 9.5	#4 @ 12		
6'-6	8	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7.5	#4 @ 9	#4 @ 12		
6'-9	8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7.5	#4 @ 9	#4 @ 12		
7'-0	8	#5 @ 8.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 8.5	#4 @ 11.5		
7'-3	8	#5 @ 8.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 8	#4 @ 11		
7'-6	8	#5 @ 8	#4 @ 6	#4 @ 6	#4 @ 6.5	#4 @ 8	#4 @ 10.5		
7'-9	8	#5 @ 7.5	#5 @ 8	#4 @ 6	#4 @ 6.5	#4 @ 7.5	#4 @ 9.5		
8'-0	8	#5 @ 7.5	#5 @ 8	#4 @ 6	#4 @ 6	#4 @ 7.5	#4 @ 9		
8'-3	8.5	#5 @ 8	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 8	#4 @ 9.5		
8'-6	8.5	#5 @ 8	#4 @ 6	#4 @ 6	#4 @ 6.5	#4 @ 8	#4 @ 9		
8'-9	8.5	#5 @ 8	#5 @ 8	#4 @ 6	#4 @ 6.5	#4 @ 7.5	#4 @ 8.5		
9'-0	8.5	#5 @ 8	#5 @ 8	#4 @ 6	#4 @ 6.5	#4 @ 7.5	#4 @ 8.5		
9'-3	8.5	#5 @ 7.5	#5 @ 8	#4 @ 6	#4 @ 6.5	#4 @ 7.5	#4 @ 8		
9'-6	9	#5 @ 8	#4 @ 6	#4 @ 6	#4 @ 6.5	#4 @ 7.5	#4 @ 8.5		
9'-9	9	#5 @ 7.5	#5 @ 8	#5 @ 8	#4 @ 6	#4 @ 7	#4 @ 8		
10'-0	9	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6	#4 @ 7	#4 @ 8		
10'-3	9	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 7.5		
10'-6	9	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 7		
10'-9	9.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 7.5		
11'-0	9.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 7		
11'-3	9.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#4 @ 6	#4 @ 7		
11'-6	9.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#4 @ 6.5		
11'-9	10	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#4 @ 7		

TRANSVERSE REINFORCING STEEL FOR DECK SLABS ON PRECAST  
CONCRETE DECK PANELS, HL93 LOADING

GIRDER SPACING "S"	TOTAL SLAB THICKNESS	DISTANCE FROM C/L OF GIRDER TO DESIGN SECTION (INCHES)					
Ft. – In.	In.	3"	4"	5"	6"	10"	15"
12'-0	10	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#4 @ 6.5
12'-3	10	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#4 @ 6.5
12'-6	10	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7.5	#4 @ 6
12'-9	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 8	#4 @ 6.5
13'-0	10.5	#5 @ 6	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8
13'-3	10.5	#5 @ 6	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8
13'-6	10.5	#5 @ 6	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8
13'-9	10.5		#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8
13'-9	11	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8.5
14'-0	11	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 8.5

Designed per AASHTO LRFD. Max. Allowable Design Stresses:  $f_c = 4$  ksi,  $f_y = 60$  ksi. Steel is 2 1/2" clear from top.  
Designed for 20 psf future wearing surface. "TOTAL SLAB THICKNESS" includes thickness of deck panel and poured in place concrete.

TABLE 17.6

## (5) Special Provisions

This standardized special provision for optional use of Precast prestressed concrete deck plans is included for informational purposes only and may not be the latest version as the STSP maintained by the Bureau of Highway Construction, Standards Development Section.

Special Provisions for Optional Use of Precast Prestressed Concrete Deck Plans

Precast prestressed concrete deck panels may be used in the construction of the concrete deck between the exterior girders, at the option of the Contractor, in lieu of construction of the deck as shown on the plans. Design and construction under this option shall be as stated in Article 17.7 of the Wisconsin Bridge Manual and as herein specified. Conventional forming shall be used for the overhang of the exterior girder.

A. Plant Certification Precast prestressed concrete deck panels shall be manufactured in a plant meeting the requirements of 503.2.4 of the Standard Specifications.

B. Materials The Precast prestressed concrete deck panels shall be manufactured in accordance with the applicable requirements of Section 503 of the Standard Specifications and as hereinafter specified.

The maximum allowable dimensional tolerances for the deck panels shall be as follows:

Thickness	+ 3/16" or -0"
Length	± 1/4"
Width	± 1/4"
Squareness (difference in diagonal lengths	1/2"
Location of Strands (measured from bottom of plank to centerline strand)	(panel thickness)/2 ± 1/8"
Bowing	± 1/8"
Sweep	± 1/8"
Warping	1/16 inch per foot of distance from nearest adjacent corner

The top surfaces of deck panels shall be given a scored finish with a tining rake or similar tool. The spacing of the scoring shall be 3/4 to 1 inches and to a depth of 1/8 to 3/16 inch. It is the intent with the scoring to obtain a roughened surface; however, areas of mortar buildup in excess of 3/8 inch above the top surface of the panel shall be removed.

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Panels having cracks visibly apparent radiating from the strand at the end of the panel will be rejected.

High-density expanded polystyrene panel supports shall have a compressive strength sufficient to support dead loads and construction loads at less than 5% deflection.

If grout is used to support the deck panels it shall be an approved non-shrink non-chloride grout mixed and placed in accordance with the grout manufacturer's instructions and specifications. The exposed grout surfaces shall be cured for 3 days using the wetted burlap method.

The welded deformed steel wire fabric shall be ASTM A497.

The panels shall have a concrete strength of 5000 psi at 28 days and a minimum release strength of 4000 psi.

All reinforcing steel placed in the poured-in-place concrete shall be coated ASTM A615 Grade 60.

C. Design The contract plans specify a reinforced concrete deck which has been designed and detailed for use of removable forms. The optional deck panel system will replace the specified bottom reinforcement bars and the concrete displaced by the panels and will eliminate the need for forms in the areas between the beams. Copings and concrete diaphragms must be formed and constructed as shown on the contract plans.

The design of the precast prestressed concrete deck panels shall be as shown in Section 17.7 of the Wisconsin Bridge Manual. The design of the upper layer of reinforcing steel in the poured-in-place concrete shall be as shown in Section 17.3 or 17.7. The size and spacing of the bottom reinforcing bars in the slab overhang are to be as shown on Figure 17.2 of the Wisconsin Bridge Manual or as specified by the Engineer. Shop plans for the deck panels and the modified concrete deck including revised shear stud spacings on steel girders when applicable, shall be submitted to the Engineer for approval.

On skewed decks, the Contractor may form and cast the skewed portion of the deck or use skewed end deck panels. The skewed end panels may be individually precast or saw-cut from square end panels. In no case, however, shall the skewed or cut end panels have less than two foot bearing length along the edge.

D. Handling and Storing The handling of precast, prestressed concrete deck panels, from the time of releasing the strands until they are in place in the structure, shall be in accordance with Section 503 of the Standard Specifications and as hereinafter specified.

The panels shall be maintained in a flat position at all times and shall be supported at approximately (15) inches from ends. The support shall be at right angles to the strands and shall extend the full width of the panel.

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During storage and transportation, the supports shall maintain the panels in essentially a level position and without twisting. When panels are stacked, the supports of all planks shall be in the same vertical planes and shall be of adequate thickness to prevent damage to the lifting hooks.

E. Construction The deck panels shall be supported as shown in Detail A, Figure 17.2 of the Wisconsin Bridge Manual. The thickness of the temporary support shall be the same as the haunch height as determined by the Engineer. If grout is used for the permanent support of the deck panels it shall be placed immediately before the placement of panels.

Prior to placement of the concrete over the panels the top of the panels shall be clean and free of any foreign material. Immediately prior to placing the concrete over the panels the top surface of the panels shall be wetted until free moisture appears and remains.

Poured-in-place concrete and reinforcement bars shall be placed in accordance with Section 501 and 505 of the Standard Specifications. Particular emphasis shall be placed on proper vibration of the concrete at the panel ends over the girders to avoid honeycombs and voids.

F. Payment If the Contractor elects to use the precast prestressed concrete deck panel system, as herein specified, payment for the deck completed and in place will be made on the basis of plan quantities and at the contract prices bid for the cast-in-place deck.

RECTANGULAR SECTION WITH TENSION REINFORCEMENT ONLY  
 STEEL YIELD STRENGTH = 60000. PSI CONCRETE COMPRESSIVE STRENGTH = 3500. PSI  
 STRENGTH METHOD

$R_u = \frac{M}{\phi b d^2 \text{ psi}}$ $e = \frac{A_s}{b d}$									
RU	RHO	RU	RHO	RU	RHO	RU	RHO	RU	RHO
117.4	0.0020	306.4	0.0054	481.1	0.0088	641.9	0.0122	788.7	0.0156
123.3	0.0021	311.7	0.0055	486.1	0.0089	646.5	0.0123	792.9	0.0157
129.1	0.0022	317.0	0.0056	491.0	0.0090	651.0	0.0124	796.9	0.0158
134.8	0.0023	322.3	0.0057	495.9	0.0091	655.5	0.0125	801.0	0.0159
140.5	0.0024	327.6	0.0058	500.8	0.0092	659.9	0.0126	805.1	0.0160
146.2	0.0025	332.9	0.0059	505.7	0.0093	664.4	0.0127	809.2	0.0161
151.9	0.0026	338.2	0.0060	510.5	0.0094	668.9	0.0128	813.2	0.0162
157.6	0.0027	343.5	0.0061	515.4	0.0095	673.3	0.0129	817.2	0.0163
163.3	0.0028	348.7	0.0062	520.2	0.0096	677.7	0.0130	821.3	0.0164
168.9	0.0029	354.0	0.0063	525.1	0.0097	682.2	0.0131	825.3	0.0165
174.6	0.0030	359.2	0.0064	529.9	0.0098	686.6	0.0132	829.3	0.0166
180.2	0.0031	364.4	0.0065	534.7	0.0099	691.0	0.0133	833.2	0.0167
185.8	0.0032	369.6	0.0066	539.5	0.0100	695.3	0.0134	837.2	0.0168
191.4	0.0033	374.8	0.0067	544.3	0.0101	699.7	0.0135	841.2	0.0169
197.0	0.0034	380.0	0.0068	549.0	0.0102	704.1	0.0136	845.1	0.0170
202.6	0.0035	385.2	0.0069	553.8	0.0103	708.4	0.0137	849.1	0.0171
208.2	0.0036	390.3	0.0070	558.6	0.0104	712.6	0.0138	853.0	0.0172
213.7	0.0037	395.5	0.0071	563.3	0.0105	717.1	0.0139	856.9	0.0173
219.3	0.0038	400.6	0.0072	568.0	0.0106	721.4	0.0140	860.8	0.0174
224.8	0.0039	405.8	0.0073	572.7	0.0107	725.7	0.0141	864.7	0.0175
230.3	0.0040	410.9	0.0074	577.4	0.0108	730.0	0.0142	868.6	0.0176
235.8	0.0041	416.0	0.0075	582.1	0.0109	734.3	0.0143	872.4	0.0177
241.3	0.0042	421.0	0.0076	586.8	0.0110	738.5	0.0144	876.3	0.0178
246.8	0.0043	426.1	0.0077	591.4	0.0111	742.6	0.0145	880.1	0.0179
252.3	0.0044	431.2	0.0078	596.1	0.0112	747.0	0.0146	884.0	0.0180
257.7	0.0045	436.2	0.0079	600.7	0.0113	751.2	0.0147	887.8	0.0181
263.2	0.0046	441.3	0.0080	605.4	0.0114	755.5	0.0148	891.6	0.0182
268.6	0.0047	446.3	0.0081	610.0	0.0115	759.7	0.0149	895.4	0.0183
274.1	0.0048	451.3	0.0082	614.6	0.0116	763.9	0.0150	899.1	0.0184
279.5	0.0049	456.3	0.0083	619.2	0.0117	768.0	0.0151	902.9	0.0185
284.9	0.0050	461.3	0.0084	623.7	0.0118	772.2	0.0152	906.7	0.0186
290.3	0.0051	466.3	0.0085	628.3	0.0119	776.4	0.0153	910.4	0.0187
295.6	0.0052	471.2	0.0086	632.9	0.0120	780.5	0.0154		
301.0	0.0053	476.2	0.0087	637.4	0.0121	784.6	0.0155		

TABLE 17.7

RECTANGULAR SECTION WITH TENSION REINFORCEMENT ONLY  
STRENGTH METHOD

STEEL YIELD STRENGTH = 60000. PSI  
CONCRETE COMPRESSIVE STRENGTH = 4000. PSI

$$R_u = \frac{M_u}{\phi b d^2}$$

$$e = \frac{A_s}{b d}$$

R <sub>u</sub>	R <sub>HO</sub>	R <sub>u</sub>	R <sub>HO</sub>	R <sub>u</sub>	R <sub>HO</sub>	R <sub>u</sub>	R <sub>HO</sub>	R <sub>u</sub>	R <sub>HO</sub>	R <sub>u</sub>	R <sub>HO</sub>	R <sub>u</sub>	R <sub>HO</sub>
117.9	0.0020	335.6	0.0059	537.1	0.0098	722.6	0.0137	892.0	0.0176				
123.7	0.0021	340.9	0.0060	542.1	0.0099	727.2	0.0138	896.1	0.0177				
129.4	0.0022	346.3	0.0061	547.1	0.0100	731.7	0.0139	900.2	0.0178				
135.2	0.0023	351.6	0.0062	552.0	0.0101	736.2	0.0140	904.4	0.0179				
141.0	0.0024	357.0	0.0063	556.9	0.0102	740.7	0.0141	908.5	0.0180				
146.7	0.0025	362.3	0.0064	561.8	0.0103	745.2	0.0142	912.5	0.0181				
152.4	0.0026	367.6	0.0065	566.7	0.0104	749.7	0.0143	916.6	0.0182				
158.1	0.0027	372.9	0.0066	571.6	0.0105	754.2	0.0144	920.7	0.0183				
163.8	0.0028	378.2	0.0067	576.5	0.0106	758.7	0.0145	924.8	0.0184				
169.5	0.0029	383.5	0.0068	581.4	0.0107	763.1	0.0146	928.9	0.0185				
175.2	0.0030	388.8	0.0069	586.2	0.0108	767.6	0.0147	932.9	0.0186				
180.9	0.0031	394.1	0.0070	591.1	0.0109	772.0	0.0148	936.9	0.0187				
186.6	0.0032	399.3	0.0071	595.9	0.0110	776.5	0.0149	940.9	0.0188				
192.2	0.0033	404.6	0.0072	600.8	0.0111	780.9	0.0150	944.9	0.0189				
197.9	0.0034	409.8	0.0073	605.6	0.0112	785.3	0.0151	948.9	0.0190				
203.5	0.0035	415.0	0.0074	610.4	0.0113	789.7	0.0152	952.9	0.0191				
209.1	0.0036	420.2	0.0075	615.2	0.0114	794.1	0.0153	956.8	0.0192				
214.8	0.0037	425.4	0.0076	620.0	0.0115	798.4	0.0154	960.8	0.0193				
220.4	0.0038	430.6	0.0077	624.8	0.0116	802.8	0.0155	964.7	0.0194				
225.9	0.0039	435.8	0.0078	629.5	0.0117	807.2	0.0156	968.7	0.0195				
231.5	0.0040	441.0	0.0079	634.3	0.0118	811.5	0.0157	972.6	0.0196				
237.1	0.0041	446.1	0.0080	639.0	0.0119	815.8	0.0158	976.5	0.0197				
242.7	0.0042	451.3	0.0081	643.8	0.0120	820.1	0.0159	980.4	0.0198				
248.2	0.0043	456.4	0.0082	648.5	0.0121	824.5	0.0160	984.3	0.0199				
253.7	0.0044	461.5	0.0083	653.2	0.0122	828.8	0.0161	988.2	0.0200				
259.3	0.0045	466.6	0.0084	657.9	0.0123	833.1	0.0162	992.1	0.0201				
264.8	0.0046	471.7	0.0085	662.6	0.0124	837.3	0.0163	996.0	0.0202				
270.3	0.0047	476.8	0.0086	667.3	0.0125	841.6	0.0164	999.9	0.0203				
275.8	0.0048	481.9	0.0087	671.9	0.0126	845.9	0.0165	1003.7	0.0204				
281.3	0.0049	487.0	0.0088	676.6	0.0127	850.1	0.0166	1007.5	0.0205				
286.8	0.0050	492.1	0.0089	681.3	0.0128	854.3	0.0167	1011.3	0.0206				
292.2	0.0051	497.1	0.0090	685.9	0.0129	858.4	0.0168	1015.1	0.0207				
297.7	0.0052	502.2	0.0091	690.5	0.0130	862.8	0.0169	1018.9	0.0208				
303.1	0.0053	507.2	0.0092	695.1	0.0131	867.0	0.0170	1022.7	0.0209				
308.6	0.0054	512.2	0.0093	699.7	0.0132	871.2	0.0171	1026.5	0.0210				
314.0	0.0055	517.2	0.0094	704.3	0.0133	875.4	0.0172	1030.3	0.0211				
319.4	0.0056	522.2	0.0095	708.9	0.0134	879.5	0.0173	1034.0	0.0212				
324.8	0.0057	527.2	0.0096	713.5	0.0135	883.7	0.0174	1037.8	0.0213				
330.2	0.0058	532.2	0.0097	718.1	0.0136	887.9	0.0175						

e MAX. = 7.52b

TABLE 17.8